FEASIBILITY STUDY APPENDICES

Quendall Terminals Site

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APPENDIX A

Groundwater Modeling

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A1 Introduction

This appendix documents the use of groundwater flow and contaminant fate and transport modeling to support the Quendall Terminals Site (Site) Feasibility Study (FS). The primary objective of FS groundwater modeling is to simulate groundwater flow and contaminant fate and transport at the Site to support the following FS tasks:

- Development and evaluation of FS remedial alternatives, including: 1) how technologies addressing groundwater contamination may be applied to achieve the preliminary remediation goals (PRGs) for one or more of the four primary chemicals of concern (COCs); 2) estimating the relative restoration timeframe; and 3) estimating the relative reduction in the volume of contaminated groundwater (groundwater plume volume), contaminant mass, and contaminant mass flux; and
- Evaluation of conceptual dewatering design, including pumping and drawdown estimates for construction dewatering, to support cost estimating.

Groundwater modeling simulations are discussed and the results evaluated in Sections 3, 6, 7, and 8 of the main text of this FS. Section 3 includes a description of the geologic conditions and hydrogeologic conceptual site model (CSM) that form the basis for the groundwater flow model. In Section 6, which assembles and describes 10 remedial alternatives, the groundwater model is used to develop conceptual design parameters such as dewatering flowrates and treatment areas. Modeling predictions of alternative effectiveness at restoring groundwater (including achieving groundwater maximum contaminant levels [MCLs], reducing the volume of contaminated groundwater, and reducing the flux of contaminants in groundwater) are used in the detailed analysis of alternatives in Section 7 and the comparative analysis of alternatives in Section 8.

A2 Groundwater Model Background

FS groundwater modeling is based on the groundwater flow and contaminant fate and transport model originally developed in support of the Site's Remedial Investigation (RI) Report (Anchor QEA and Aspect 2012). The groundwater model is a MODFLOW-based (MacDonald and Harbaugh 1988), three-dimensional numerical model of groundwater flow across the Site. The groundwater model uses the code MT3DMS (Zheng and Wang 1999), an update to the original MT3D code (Zheng 1990), to simulate contaminant fate and transport. Documentation of the construction and calibration of the groundwater model used to support preparation of the RI Report is provided in Appendix D of the RI Report (Anchor QEA and Aspect 2012).

The original RI groundwater model that was developed and described in Appendix D of the RI Report (Anchor QEA and Aspect 2012), has been refined and used for two general purposes in the FS:

1. Development and evaluation of FS remedial alternatives. This FS groundwater modeling task used modifications to both the groundwater flow and contaminant fate and transport components from the RI groundwater model to produce the groundwater model results described in Section A3 of this appendix.
2. Evaluation of FS conceptual dewatering design criteria. This FS groundwater modeling task used modifications to only the groundwater flow component of the RI groundwater model to produce the groundwater model results described in Section A4 of this appendix.

The groundwater model structure, groundwater flow boundary conditions, and flow parameters used to perform groundwater modeling tasks in the FS remain unchanged from those used in the RI groundwater model with the following exceptions: modifications to the grid to increase vertical resolution and the addition and/or modification of boundary conditions and parameters to simulate elements of remedial alternatives or dewatering systems consistent with the description of the alternatives presented in Section 6 of the FS. The specific structural modifications to the groundwater model used to evaluate FS remedial alternatives and determine FS dewatering design criteria are detailed in Sections A3 and A4 of this appendix, respectively.

Several groundwater modeling evaluations specific to Alternatives 9 and 10 were completed early in the FS process using slightly different groundwater model assumptions and construction than the analyses described in Sections A3 and A4. Differences in the groundwater model include different initial concentration conditions and local grid discretization. These earlier evaluations included optimizing the conceptual design of a pump and treat polishing system for Alternative 9, determining construction dewatering design criteria for Alternatives 9 and 10, and evaluating the potential effect of Deep Aquifer heterogeneity and potential excavation residuals on restoration timeframe on Alternative 10. For these analyses, we do not expect that the differences in the groundwater model construction significantly affect the results or conclusions; therefore, these earlier evaluations were not reanalyzed using the updated groundwater model described in Sections A3 and A4. Groundwater model construction and results for these earlier evaluations are described in more detail in Section A5.

A3 Evaluation of Remedial Alternatives

The FS groundwater model was used to simulate changes in concentrations of the four primary COCs (benzene, benzo(a)pyrene[1], naphthalene, and arsenic; refer to Section 3.5 of the FS main text) in Site groundwater following implementation (i.e., completion of construction) of individual remedial alternatives. The groundwater modeling approach used for this evaluation was a four-step process as follows:

1. In the first step, the distribution of dense non-aqueous phase liquid (DNAPL) observed at the Site (Section 3 of the FS main text) was represented as a source of contamination in the groundwater model by placing constant groundwater concentration boundary conditions (based on existing Site data) in the groundwater model cells corresponding to DNAPL-impacted soil.

2. In the second step, the groundwater model was run for 100 years to simulate the time since the creosote plant began operation, and to “propagate” the dissolved phase plumes. The propagated plumes were used to generate an approximate representation of the Site’s downgradient pre-remediation concentration distributions for each of the three primary hydrocarbon COCs (benzene, benzo[a]pyrene, and naphthalene) derived from the hydrocarbon source2.

3. In the third step, the hydrocarbon source (constant groundwater concentration boundary condition) and the pre-remediation concentrations of each of the four primary COCs were modified to reflect implementation of the remedial alternative being evaluated to generate a post-remedy initial condition and boundary conditions for each of the alternatives. For example, for an area where DNAPL would be removed as part of an alternative, the hydrocarbon source (constant groundwater concentration boundary condition) was removed and the pre-remediation concentrations of each of the four primary COCs were set to zero (conservatively assuming no residual soil or groundwater contamination remaining following remedial construction).

4. In the fourth step, the FS groundwater model was then run using those post-remedy boundary conditions and initial conditions for an additional 100 years to predict the groundwater concentrations of the primary COCs, 100 years following completion of construction of the remedial action.

This groundwater modeling evaluation is intended to be used as a predictive tool to provide relative results based on a consistent set of assumptions for comparative evaluation of the range of remedial alternatives. Simplifying assumptions were made in order to represent the complexities of Site conditions in the groundwater model and simulate the transport of the primary COCs. Because of the simplifying assumptions, the

[1] Benzo(a)pyrene is modeled to represent total carcinogenic polycyclic aromatic hydrocarbons (cPAHs; as a benzo[a]pyrene equivalent).
[2] The term “hydrocarbon source” is specific to the groundwater model. “DNAPL source” is a more general term and is used in the main text.

groundwater model results should be viewed as an approximate representation of actual outcomes (see Section A3.2.3 for examples that illustrate the differences between modeled and actual conditions). Therefore, results should be used to compare the relative benefit of different alternatives rather than as absolute predictions of actual outcomes.

The sections listed below and that follow, describe construction and use of the FS groundwater model in the evaluation of the remedial alternatives:

- Section A3.1 describes the modifications to the RI groundwater model used to develop the FS groundwater model;
- Section A3.2 describes the methods used to establish contaminant fate and transport boundary conditions and initial conditions;
- Section A3.3 details the alternative-specific modifications to the groundwater model to evaluate the effect of different remedial technologies;
- Section A3.4 describes groundwater modeling conducted to aid alternative development;
- Sections A3.5 describes simulation of the alternatives;
- Section A3.6 describes results of the alternative evaluation; and
- Section A3.7 documents the sensitivity analysis.

A3.1 Modifications to Develop FS Groundwater Model

The following modifications were made to the groundwater flow and contaminant fate and transport components of the RI groundwater model to develop the FS groundwater model. The modifications include both structural modifications and updates to contaminant fate and transport parameters as described below.

A3.1.1 Structural Modifications

Structural modifications were made to the RI groundwater model to facilitate its use for evaluation of remedial alternatives in the FS. The groundwater model developed for the FS includes inserting 19 additional layers to increase vertical resolution for simulation of remedial alternatives that include solidification (Alternatives 3, 5, 6, 7, and 9). Eight additional groundwater model layers were added by evenly splitting layers 3 (top of the Shallow Aquifer) through 10 (deepest layer) of the RI groundwater model in half. The top layer of the Deep Aquifer was then subdivided by adding two, 2- to 3-foot-thick,layers at the top of the Deep Aquifer to facilitate simulation of DNAPL at the top of the Deep Aquifer. The geometry of hydrostratigraphic zones and groundwater model boundaries were unchanged. The grid change was applied to all remedial alternatives to maintain consistency for volume calculations.

A3.1.2 Transport Parameter Modifications

The contaminant fate and transport parameters for the hydrocarbon primary COCs used in the FS groundwater modeling were consistent with assumptions used in the RI groundwater model. For the FS analyses, transport of arsenic was added, assuming a sorption coefficient (K_d) of 29 liters per kilogram (L/kg) as presented in the Washington State Department of Ecology (Ecology) Model Toxics Control Act (MTCA) regulation

(Washington Administrative Code [WAC] 173-340-900 Table 747-3). Arsenic decay was not simulated because arsenic does not decay. These parameters are presented in Table A-1. While parameters remained unchanged from the RI version of the groundwater model, a few select parameters were re-evaluated in detail to ensure they satisfy the purposes of the FS; those evaluations are discussed in the sections below.

A3.1.2.1 Contaminant Degradation

The value used for the half-life of benzene was re-evaluated because the half-life has a large effect on the groundwater model results as is shown in the sensitivity analysis (Section A-3.7). As described below, benzene likely undergoes biodegradation in Site groundwater under anaerobic conditions. A benzene half-life value of 720 days was used in both the RI and FS groundwater models and is our best estimate of anaerobic degradation of benzene on the Site. This best estimate and the range of half-lives used in the sensitivity analysis are consistent with those in applicable published literature under anaerobic conditions. A review of Site groundwater conditions, a summary of half-lives used in previous Site evaluations, and a discussion of the half-life values used in the FS based on an updated literature review are provided below.

- **Review of Groundwater Conditions.** At low dissolved oxygen (DO) levels (e.g., 1 mg/L), anaerobic respiration is the dominant biodegradation mechanism (Aaronson 1997). Site RI data show very low DO concentrations (average of 0.77 mg/L for all wells sampled in 2008 and 2009[3]; see Table A-2) that are consistent with anaerobic conditions. Other groundwater conditions observed at the Site, such as elevated dissolved iron, also indicate anaerobic conditions. Anaerobic conditions are common at sites containing significant sources of organic carbon, which serve as a food source for indigenous bacteria. At the Site, both natural (e.g., peat) and anthropogenic (e.g., DNAPL) sources of organic carbon are present.

- **Summary of Half-Lives Used in Previous Site Evaluations.** Previous transport modeling of the Site by Retec 1998 used column testing results reported in a treatability study (Retec 1997) and literature values reported in the Handbook of Environmental Degradation Rates (Howard et al. 1991) as a basis for degradation rates, as follows:
  - Aerobic Conditions: Retec modeled degradation for an aerated treatment system using a range of half-life values based on aerated column testing results and aerobic rates reported in Howard et al. 1991. The test protocol for the treatability study column testing was designed to simulate conditions representative of the peak performance achievable from an aeration system; therefore, influent DO concentrations to the test columns were maintained at 6 mg/L.
  - Anaerobic Conditions: Anaerobic benzene half-lives considered by Retec 1998 were based on the values reported in Howard et al. 1991, ranged

[3] Data collected in 2008 and 2009 are considered most representative of Site conditions for two reasons: 1) they are the most recent available data and 2) some of the older groundwater data was collected using bailers, which can bias DO measurements high.

from 112 to 720 days. However, Retec assumed no degradation for model simulations that represented no aeration.

Half-lives based on the Retec treatability study column test results are not representative of current conditions at the Site because DO measured on site is far less than what would be expected under aerated conditions. Therefore the values resulting from column testing were not considered for the RI or FS groundwater model.

- **FS Half-Lives Values Used Based on Updated Literature Review.** For the FS, an updated review of the literature for anaerobic biodegradation was conducted. A more extensive review of laboratory and field studies is provided in Aronson 1997. This review indicated anaerobic half-lives for benzene determined under field studies ranged from 220 days to no degradation.

  The longer of the anaerobic half-life values reported in Howard et al. 1991 (720 days for benzene) was selected as an appropriate mid-range value for the FS groundwater model, based on the range of half-life values derived from representative field studies (Aronson 1997). The 720-day half-life is the same that was used in fate-and-transport modeling for the RI Report (Anchor and Aspect, 2012).

  A sensitivity analysis was also performed (discussed in Section A3.7) which included a shorter half-life for benzene (112 days) that is based on laboratory studies (Howard et al. 1991). This value is lower than the shortest half-life rate (220 days) derived from field studies reviewed by Aronson 1997.

  Representative half-life values for naphthalene and benzo(a)pyrene where derived similarly, as follows:

  - Retec 1998 assumed values for chrysene were representative of benzo(a)pyrene. This assumption was retained for the RI and FS groundwater models. The longer anaerobic half-life reported for chrysene in Howard et al. 1991 (4,000 days) was assumed for benzo(a)pyrene in the FS groundwater model; the shorter anaerobic half-life for chrysene reported in Howard et al. 1991 (1,484 days) was used as the lower bound in the sensitivity analysis.
  - Only one anaerobic half-life for naphthalene was reported in Howard 1991 (258 days); therefore, this value was used for the FS groundwater transport modeling. To arrive at the lower bound for the naphthalene half-life sensitivity analysis (40 days), the naphthalene half-life was reduced an amount proportional to the reduction of the benzene half-life (84 percent).

Fill Sorption Coefficient (Kd)

The sorption coefficient (K_d) parameter defines sorption processes in the groundwater transport model, and K_d values used in the groundwater model are based on the fraction of organic content (f_{oc}) assumed in each hydrostratigraphic unit. The groundwater model uses the same K_d in the fill as in the Deeper Alluvium. Previous modeling at the Site (Retec 1998) assumed a K_d value in the fill that is equal to the value in the Shallow

Alluvium because a higher f_{oc} was assumed in the fill due to the presence of woody debris. While a K_d value based on a higher f_{oc} may be more appropriate for some materials in the Fill Unit, the difference in K_d in the fill is expected to have a *de minimis* effect on the groundwater model results because the fill is only partially saturated and the saturated fill makes up a very small portion of the active model domain. Therefore, the K_d value used in the RI groundwater model was retained in the FS groundwater model and considered adequate for the purposes of the FS.

A3.1.3 Simplifying Assumptions

The FS groundwater model makes two simplifying assumptions that were evaluated for the FS. These assumptions are as follows:

1. Homogeneous Deep Aquifer. The groundwater model assumes the Deep Aquifer is homogeneous when in actuality; it contains lenses of lower permeability material where higher concentrations may persist for a longer duration than what the groundwater model predicts for the assumed homogeneous materials.
2. No Excavation Residuals. When simulating excavation of contaminated soil, the groundwater model assumes that no residual groundwater or soil contamination remains in the excavated volume after construction. Groundwater model simulation of excavations is discussed in Section A3.

These assumptions were also included in the RI version of the groundwater model and were not modified; however, they are mentioned here because they may have a significant effect on the potential for the most aggressive alternatives to achieve drinking water MCLs for Site COCs, and the results of the evaluation of remedial alternatives should be considered with this in mind. Any contribution to concentrations from fine-grained layers or excavation residual would be in addition to the groundwater-model-predicted concentrations resulting from this remedial alternative evaluation presented in Section A3.6. The effect of these simplifying assumptions on the groundwater model results was evaluated in a sensitivity analysis conducted during groundwater modeling early in the FS process, as discussed in Section A5. Based on the sensitivity analysis, the FS groundwater model likely underpredicts restoration timeframes for recalcitrant compounds (e.g., benzo(a)pyrene and arsenic) and therefore should be viewed as a best case scenario. The restoration timeframe for more easily degraded compounds (e.g., benzene) is less sensitive to these parameters.

A3.1.4 Modifications to Simulate Remedial Actions

The contaminant fate and transport parameters discussed above are intended to simulate current Site conditions. In some cases however, contaminant fate and transport parameter values were changed specific to individual remedial technologies as described below in Section A3.3: Groundwater Model Simulation of Remedial Technologies and Section A3.5: Groundwater Model Simulation of Remedial Alternatives. Changes to contaminant fate and transport parameters as part of the sensitivity analysis are described in Section A3.7.

A3.2 Initial Conditions and Hydrocarbon Source Boundary Conditions for FS Remedial Alternatives

Initial conditions for each remedial alternative groundwater model run were developed to represent concentrations of the four primary COCs throughout the Site immediately following implementation of the alternative (see Section 6 of the FS main text for detailed descriptions of each alternative).

These initial conditions are specific to each alternative and vary depending on how implementation of an alternative is expected to alter the pre-remediation concentrations. This subsection describes the manner in which the pre-remediation (present day) concentrations were established and how they were then modified to establish post-remedy initial conditions (i.e., represent Site conditions immediately following completion of construction of the remedial action) for each alternative.

A3.2.1 Initial Conditions and Source Boundary Conditions for Benzene, Benzo(a)pyrene, and Naphthalene

Source propagation was used when possible to define the initial condition following implementation of a remedial alternative for two reasons: 1) to address the variability of observed (empirical) dissolved phase concentrations and uncertainty in concentration distribution across the Site and 2) because it provides a consistent basis for comparing remedial alternatives. When initial conditions are simply assigned and not generated by the groundwater model, the subsequent simulated transport can be largely a result of the initial conditions readjusting to fit the transport field and source distribution. These adjustments are difficult to parse out from the changes to concentrations caused by the stresses on the groundwater model that represent remedial technologies, especially when sources remain in the alternative being simulated. Pre-remediation concentrations for the DNAPL-related primary COCs (benzene, benzo(a)pyrene, and naphthalene) were generated with simulated hydrocarbon sources within the groundwater model based on the distribution of the hydrocarbon source (DNAPL). Because hydrocarbon sources are left in place in many of the alternatives, groundwater-model-propagated pre-remediation concentrations provide a better relative comparison of plume reduction. Pre-remediation hydrocarbon concentrations were generated using the following methodology:

- The pre-remediation dissolved and sorbed soil concentrations for benzene, benzo(a)pyrene, and naphthalene were produced with constant groundwater concentration boundary conditions representing DNAPL as hydrocarbon sources. The Thiessen polygon distribution of DNAPL depth and lateral extent (depicted on Figure 4.4-5 of the RI Report and on Figure A-1) was used to define hydrocarbon-source zones in the FS groundwater model.
- Values for the constant groundwater concentration boundary conditions for benzo(a)pyrene[4] and naphthalene were assumed to be the average of concentrations detected in groundwater from Shallow Alluvium monitoring wells and groundwater grab samples in DNAPL-impacted areas (BH-19, BH-21A, BH-

[4] Total cPAH concentration as benzo(a)pyrene equivalent were used to calculate the benzo(a)pyrene boundary condition concentration.

25A(R), BH-20A, BH-5, BH-23, RW-NS-1, RW-QP-1, and Q9[5]) and reported in the RI Report (Anchor QEA and Aspect 2012). Average concentrations were 133 micrograms per liter [µg/L] for benzo(a)pyrene and 11,000 µg/L naphthalene (see Table A-3).

Values for the constant concentration boundary conditions for benzene were also assumed to be the average concentration detected in DNAPL-impacted areas, but were separated into the following five different zones to reflect spatial variability:

- Zone 1 includes well BH-21A (average concentration of 4 µg/L, but benzene was not simulated in this boundary condition because the concentration is exceeded by nearby plume concentrations; if simulated, the boundary condition would artificially remove benzene mass from the aquifer);
- Zone 2 includes Wells BH-25A(R) and Q9 (average concentration of 1,100 µg/L);
- Zone 3 includes well Q-14W (benzene was not detected; therefore, benzene was not simulated in this zone);
- Zone 4 includes wells BH-23 and RW-NS1 (average concentration of 200 µg/L); and
- Zone 5 includes wells BH-5, BH-19, BH-20A, and RW-QP1 (average concentration of 12,000 µg/L).

Associated solid-phase concentrations were calculated by the groundwater model by applying the respective Kd values and assuming equilibrium. Figure A-1 shows the distribution and concentration of the hydrocarbon sources. Data used to produce these estimates are summarized in Table A-3. Figures A-2 and A-3 depict the source boundary conditions as implemented in the groundwater model.

- The groundwater model was then run for 100 years to simulate the time since the creosote plant started operation.

After establishing pre-remediation conditions, the resulting pre-remediation dissolved and sorbed concentrations for each of the DNAPL-related COCs were then altered consistent with the alternative being simulated and imported as the initial condition. Changes to hydrocarbon source constant groundwater concentration boundary conditions were also made consistent with the alternative being simulated. Adjustments to concentrations and boundary conditions for each of the remediation technologies are described in Section A3.3.

A3.2.2 Initial Conditions for Arsenic

No soil source of arsenic has been identified at the Site so it is not possible to generate pre-remedial arsenic concentrations by source propagation; therefore, pre-remedial concentrations for arsenic were identified based on groundwater data reported in the RI Report (Anchor QEA and Aspect 2012). The average arsenic concentration in areas

[5] The benzo(a)pyrene concentration at Q9 was excluded from averaging because the concentration exceeds solubility.

exceeding the arsenic MCL (39 µg/L)[6] was input as the pre-remediation concentration in the groundwater model. The lateral extent of the arsenic plume in the Shallow Aquifer was limited to the extent shown on Figure 5.2-16 of the RI Report (Anchor QEA and Aspect 2012). Similarly, the lateral extent in the Deep Aquifer was limited to the extent shown on Figure 5.2-17 of the RI Report (Anchor QEA and Aspect 2012). The bottom of the simulated arsenic plume is approximately 60 feet below ground surface (bgs), based on the groundwater data from well BH-20B and BH-20C. Pre-remediation concentrations outside of the arsenic plume are set to the Puget Sound-area background concentration of 5 µg/L as specified by Ecology (Ecology 2001; Table 720-1). Solid-phase concentrations were input into the groundwater model by applying the K_d value of 29 L/kg and assuming equilibrium.

The resulting pre-remediation dissolved and sorbed concentrations for arsenic were then altered relative to the alternative being simulated and imported as the initial condition to the groundwater model.

A3.2.3 Comparison to Measured Concentrations

Figures A-4 through A-7 compare, in plan view, groundwater-model-generated pre-remediation plume extents to the plume extents presented in the RI and summarized in Section 3 of the FS. Groundwater model-generated plume extents are similarly compared in cross section on Figures A-8 through A-11. Plume extents presented in Section 3 are based on a combination of empirical data with groundwater modeling and professional judgment where data is limited (as described in Section 3 of the main text). In particular, limited data are available to define the vertical extent of contaminant plumes in the Deep Aquifer and the westward extent of plumes beneath the lake, which correspond to the areas where the groundwater model predictions deviate the most from the plumes estimated for the RI. Main differences include the following:

- The groundwater model predicts the benzene and naphthalene plumes extend farther west than estimated in Section 3. The extents in Section 3 were based on available sediment porewater data (collected from shallow sediments) and predicted groundwater flow paths, but did not consider the potential effect of dispersion (which would increase the plume extent) or biodegradation (which could decrease plume extent). No data is available in deep groundwater offshore; therefore, there is uncertainty in the actual extent of the plumes in the area between the inner harbor line and the T-Dock.
- The groundwater model predicts the benzo(a)pyrene plumes do not extend as far west as estimated in Section 3. This prediction is likely due to the fact that the westerly extent in the Section 3 was based on empirical data in shallow offshore sediments, but the groundwater model did not include DNAPL in shallow offshore sediments as source terms.
- The groundwater model predicts that the vertical extent of benzo(a)pyrene in the BH-30C area is greater than estimated in Section 3. There is uncertainty in both estimates. Groundwater model uncertainties result from groundwater model simplifications (e.g., coarse vertical discretization of the Deep Aquifer with a

[6] The overall average concentration was used for simplicity because the average concentration in the Shallow and Deep Aquifers (29 and 47 µg/L, respectively) were similar.

layer thickness of approximately 10 feet), and uncertainties in groundwater model parameters (e.g., the magnitude of vertical dispersivity). Empirical data in this area is limited: DNAPL (the assumed source of benzo(a)pyrene) is present at a depth of 33.75 feet, and the top of the well screen for BH-30C is at a depth of 85 feet. As described in Section 3.5 of the FS, the vertical extent of benzo(a)pyrene at this location was estimated based on soil data from the Shallow Alluvium, which identified elevated concentrations of benzo(a)pyrene in soil up to 7 feet below DNAPL occurrences. Based on this data, the Section 3 estimated vertical extent of benzo(a)pyrene was based on adjusting the groundwater modeled extent to extend a maximum of 7 feet below the deepest DNAPL occurrences.

The groundwater model incorporates simplifying assumptions to represent the complex Site conditions including assumptions of geology, contaminant distribution, and dispersivity and degradation parameters. During groundwater model calibration, some groundwater model parameters were adjusted to more closely match the groundwater model output with empirical data for COC concentrations. However, it was not possible to match all empirical data. For example, varying dispersivity to account for naphthalene detected at deep well BH-20C resulted in the groundwater model over predicting benzene concentrations at the same well. Due to the complexity of subsurface conditions at the Site, the groundwater model results only approximate the observed (empirical) groundwater concentration distribution.

The groundwater model is meant to be used as a relative tool, meaning it is intended to compare the relative effect of different remedies, and the relative effectiveness of remedial options to reduce plumes and restoration timeframe. As described above in Section A3.2.1, setting initial conditions in the groundwater model using source propagation provides a more realistic groundwater model of contaminant distribution between areas, and the relative effect of different remedial actions on contaminant distribution are more apparent.

Necessary groundwater modeling simplifications result in differences between groundwater model predictions and actual conditions; however, we do not expect these differences to significantly affect the comparative evaluation of alternatives. While the absolute numbers such as predicted plume volume or contaminant mass should be considered approximate, the relative effect of different actions on reducing plume volume and contaminant mass are valid. Groundwater model results are meant to be interpreted in a relative manner as a means to compare the remediation potential of the different alternatives.

A3.3 Groundwater Model Simulation of Remedial Technologies

Each remedial alternative is composed of a combination of one or more of the following remedial technologies[7]:

- Impermeable upland cap;

[7] Technologies with no significant effect on groundwater flow or contaminant fate and transport in groundwater (e.g., sediment capping) were not simulated by the groundwater model.

- Funnel and gate treatment wall;
- DNAPL/soil solidification;
- DNAPL/soil excavation; and
- Pump and treat.

A detailed description of each of these technologies and how they would be applied is presented in Sections 5 and 6 of this FS. The remedial technologies are simulated within the groundwater model by modifying flow and transport parameters, and/or boundary conditions. In some cases, new boundary conditions were specified to simulate structural elements of the technologies (i.e., slurry walls). Modifications specific to each remedial technology include the following:

- **Impermeable Upland Cap.** An impermeable cap is assumed in the uplands because future development is expected to result in reduced recharge in the uplands as described in Section 6.2 of the FS. The cap is simulated in the groundwater model with a recharge boundary condition value equal to 0 inches/year.
- **Funnel and Gate Treatment Wall.** A funnel and gate system consists of two structures: a slurry wall along the shoreline and two 100-foot-long permeable reactive barriers (PRBs). The funnel and gate extends from the ground surface to approximately 30 feet bgs. The slurry wall element of the funnel and gate was simulated in the groundwater model using MODFLOW's wall boundary condition available in the horizontal flow barrier (HFB) package (Hsieh and Freckleton 1993). The wall boundary condition simulates groundwater flow barriers by applying a specified horizontal conductance (horizontal hydraulic conductivity multiplied by flow length) value between groundwater model grid cells. The conductance of the slurry walls in the funnel and gate was set at 8.5 x 10^{-4} feet squared per day (ft^2/day) to simulate a 3-foot-thick wall with a horizontal hydraulic conductivity of 2.8 x 10^{-3} feet/day (1.0 x 10^{-6} centimeters/second [cm/sec]). The PRBs were simulated in the groundwater model using a constant concentration boundary condition set to the COC-specific PRG (5 µg/L for benzene, 0.2 µg/L for benzo(a)pyrene, and 1.4 µg/L for naphthalene; arsenic is not treated). Use of the constant concentration boundary condition allows mass in excess of the PRG to be removed from the groundwater model, thereby simulating concentration reduction to PRG levels consistent with PRB design.

  The horizontal hydraulic conductivity of the PRB was scaled (16.56 ft/day) to simulate a 3-foot-thick PRB with a horizontal hydraulic conductivity of 28 feet/day (1.0 x 10^{-2} cm/sec) in the 25-foot-wide groundwater model cell.
- **DNAPL/Soil Solidification.** This technology reduces leaching of dissolved DNAPL-related COCs by physically mixing DNAPL and soil with low-permeability grout. This reduces the hydraulic conductivity of soil. Solidification was simulated in the groundwater model by changing the hydraulic conductivity, and porosity of groundwater model grid cells within the solidified volume. Based on commonly reported values for grout and clay in literature (Yey et al. 2000) and typical values for solidified soil at remediation sites (EPA 2009), hydraulic

conductivity and porosity was specified at 2.8 x 10^{-3} feet/day (1.0 x 10^{-6} cm/sec) and 0.06, respectively. The effective porosity value specified is 0.06 and is based on measured effective porosity of bentonite reported in the literature (Yey et al. 2000). The hydraulic conductivity for solidified soil was specified at 2.8 x 10^{-3} feet/day (1.0 x 10^{-6} cm/sec) based on typical values for solidified soil at remediation sites which ranged from 1 x 10^{-5} to 2 x 10^{-7} cm/sec (EPA 1999,EPA 2009, EPRI 2007, and Wilk 2007). For comparison, Table A-4 presents a summary of representative sites where solidification was implemented to contain creosote and coal tar along with the hydraulic conductivities achieved in the solidified soils.

- **DNAPL/Soil Excavation.** The excavation of DNAPL and soil was simulated by removing constant concentration boundary conditions representing DNAPL from groundwater model grid cells within the excavation. To simulate the clean backfill, the hydraulic conductivity of excavated groundwater model grid cells was altered and their sorbed and dissolved initial conditions were set to a concentration of 0 µg/L for all COCs. As discussed in Section A3.1.3, this assumes there are no residual soil and groundwater concentrations. Actual background concentrations would vary based on backfill type and groundwater chemistry. If backfilled soil contributes arsenic to groundwater, or if benzo(a)pyrene in groundwater from neighboring excavation cells recontaminates excavated areas, the restoration timeframe would be longer.

  Initial conditions outside the excavated area were unchanged from pre-remediation levels. The excavations were backfilled with one of two types of material, as follows:

  1. If excavated in the wet, gravel backfill was placed below the water table with an assumed horizontal conductivity of 28 feet/day (1.0 x 10^{-2} cm/sec); or

  2. If excavated in the dry and the excavated soil is treated and used as backfill, then the fill was assumed to have a horizontal conductivity of 0.28 feet/day (1.0 x 10^{-4} cm/sec).

  The ratio of horizontal to vertical conductivity was assumed to be 10:1 for both types of backfill.

- **Pump and Treat.** The pump and treat system assumed six wells pumping at an individual rate of 15 gallons per minute (gpm). The wells were placed along the shoreline and screened near the top of the Deep Aquifer approximately 30 to 50 feet bgs. The techniques used to model the configuration and pumping rates of this system are the result of groundwater-model-aided optimization performed early on in the FS process. Pump and treat optimization is described in Section A5.2.

A3.4 Development of FS Remedial Alternatives

The FS groundwater model was used in the development of remedial alternatives. Additional documentation of the development of remedial alternatives is provided in

Sections 5 and 6 of the FS main text. Specific uses of the groundwater model for alternative development included the following:

- **RR DNAPL Area Treatment.** The FS groundwater model was used to compare the effectiveness of solidification versus excavation (removal) of DNAPL on the plume volume to inform development of Alternatives 3, 5, and 6. The three comparison scenarios are as follows:
  - **Comparison of Backfill Materials.** Excavation of DNAPL in the RR DNAPL Area (Area 1) with off-site disposal of soil and replacement with clean imported fill is compared to on-site treatment and backfill with treated soil.
  - **Comparison of Remedial Technologies and Treatment Areas**. For the RR DNAPL Area, *in situ* solidification was compared to excavation, on-site treatment, and backfill with treated soil. Solidification and excavation of different area combinations for more extensive treatment beyond the RR DNAPL Area were also evaluated to determine the resulting effect on groundwater restoration, as described in Section 6.3.3.1 of the FS main text. Areas evaluated are listed in Table A-5 and shown on Figure A-12. Estimated plume volume reductions resulting from these comparisons are summarized in Table A-5.
  - **Pump and Treat Optimization**. The conceptual design of the pump and treat system for Alternative 10 was developed early in the FS process and is documented in Section A5. The effectiveness of this pump and treat system to reduce restoration timeframes in Alternative 10 was evaluated by comparing the restoration timeframes of the optimized pump and treat system with two variations: one with the pump and treat system removed and one with an additional well located in the area with the highest post remediation concentration (located beneath deep DNAPL-impacted soil in the RR DNAPL Area). The resulting restoration timeframes of benzene and naphthalene were compared.

When compared to no pumping, optimized pump and treat is predicted to accelerate the restoration of naphthalene by 10 years and to have no effect on benzene restoration[8].

The differences between the effect of pump and treat on the restoration timeframe of benzene compared to naphthalene are due to the smaller half-life used for benzene. A greater proportion of benzene is removed by degradation rather than flushing and so its restoration timeframe is less sensitive to pump and treat.

Additional pumping from concentration hotspots is not estimated to provide additional benefit. When the additional pump and treat well was added to the hot spot, the resulting restoration timeframe was 14 years and 46 years for benzene and naphthalene, respectively. Both are within the 3-year printing resolution of the groundwater model when compared to the groundwater model results using the optimized pump and treat system (Table A-7).

[8] With the pump and treat system removed, the benzene restoration timeframe was reported at 13 years, 1 year less than the restoration timeframe result under optimized pumping. However, the difference is within the resolution of the groundwater model output (3 years).

- Funnel and Gate Optimization. Multiple lengths of the PRB gates in Alternatives 3 through 6 were evaluated using the FS groundwater model to verify that the length of the gates would not create significant groundwater mounding. The evaluation concluded that two 100-foot-long gates limited groundwater mounding to several feet below ground surface, with a maximum mounding height of 1.5 feet. In addition, a maximum groundwater flow velocity of 1.1 feet/day was simulated through the gate, occurring in the Fill Unit. This groundwater flow velocity was used to inform the PRB design (see Appendix E of this FS for details).

  The potential for the funnel to induce lateral spreading of groundwater contamination was also evaluated. The potential for lateral spreading was determined not a risk as demonstrated by the simulated plumes for Alternatives 3 through 6, which are shown on Figures A-13 through A-17; the simulated plumes do not show an expanded lateral extent relative to current conditions (Alternative 1 – No Action).

- **Potential Spreading Induced by Soil Solidification.** The potential for soil solidification to induce spreading of groundwater contamination was evaluated. The potential for lateral spreading was determined not to be a risk as demonstrated by the simulated plumes for Alternatives 3, 5, 6, 7, and 9 relative to Alternative 1 (see Figures A-13 through A-17); the simulated plumes do not show an expanded lateral extent relative to No Action conditions (Alternative 1). The simulated plumes for these alternatives along cross sections (see Figures A-18 through A-21) also show no significant vertical spreading of contamination relative to No Action (Alternative 1) conditions. Because the extent of plume spreading was not significant, potential mitigation components for spreading (e.g., upgradient drains) were not evaluated with the groundwater model.

A3.5 Groundwater Model Simulation of Remedial Alternatives

This section details the combination of remedial technologies and how the modifications described above in Section A3.3 were incorporated into the FS groundwater model to simulate each of the remedial alternatives. Once the initial conditions were established to reflect Site conditions following completion of remedial construction for each alternative, the groundwater model was then run for a 100-year period to predict the change in groundwater concentrations for the primary COCs over that period of time. The remedial alternatives are as follows:

- **Alternative 1.** Alternative 1 assumes no remedial action occurs at the Site. Pre-remediation groundwater model results and pre-remediation arsenic concentrations were input unaltered as initial conditions and no other changes to the groundwater model were made.
- **Alternative 2.** Alternative 2 includes an impermeable cap applied to the upland portion of the Site, excluding the 100-foot-wide habitat area along the shoreline (shown on Figure 6-1 of the FS main text).

- **Alternative 3.** Alternative 3 includes the impermeable cap, a funnel and gate treatment wall, and solidification of deep DNAPL-impacted soil in the RR DNAPL Area and in the vicinity of MC-1.
    - All groundwater model grid cells simulating DNAPL-impacted soil within the zone shown on Figure 6-4 of the FS main text were assumed to be solidified. In addition, a one-cell buffer (approximately 25 feet) around the zones and an approximate 2-foot-thick layer below the zones was solidified.
    - The funnel and gate design shown on Figure 6-4 was replicated in the groundwater model in model layers 2 (ground surface) through 8 (approximately 30 feet bgs). The geometry, as specified in the FS groundwater model, differs slightly from the feature shown on Figure 6-4 to fit the resolution of the groundwater model grid.
    - The impermeable cap was applied to Site uplands, but excluded the 100-foot-wide habitat area along the shoreline (shown on Figure 6-1 of the FS main text).

- **Alternative 4.** Alternative 4 includes the impermeable cap, a funnel and gate treatment wall, and excavation and removal of DNAPL-impacted soil in the Quendall Pond Upland (QP-U) DNAPL Area.
    - The funnel and gate design shown on Figure 6-7 of the FS main text was replicated in the groundwater model in model layers 2 (ground surface) through 8 (approximately 30 feet bgs). The geometry, as specified in the FS groundwater model, differs slightly from the feature shown on Figure 6-7 to fit the resolution of the groundwater model grid.
    - The footprint of the excavation in the groundwater model follows the design shown for Alternative 4 on Figure 6-7 of the FS main text and extends approximately 19 feet deep. Similarly, the geometry is slightly different from the design to fit the groundwater model grid. Backfill is assumed to be gravel with relatively high hydraulic conductivity (1.0×10^{-2} cm/sec).
    - The impermeable cap was applied to Site uplands, but excluded the 100-foot-wide habitat area along the shoreline (shown on Figure 6-1 of the FS main text).

- **Alternative 5.** Alternative 5 includes the impermeable cap and funnel and gate, with the addition of solidification of soil containing 4 or more feet (cumulative thickness) of DNAPL to a maximum depth of 20 feet bgs, the QP-U DNAPL Area, and all deep DNAPL-impacted soil in the RR DNAPL Area and in the vicinity of MC-1.
    - The following DNAPL zones were solidified: All groundwater model cells within the shallow DNAPL zones shown on Figure 6-10 of the FS main text were solidified to a depth of approximately 20 feet bgs and groundwater model cells within deep DNAPL zones were solidified to 2 feet below the DNAPL. In addition, a one-cell buffer (approximately 25 feet) around all the treated zones was solidified.

  - The funnel and gate design shown on Figure 6-10 was replicated in the groundwater model in model layers 2 (ground surface) through 8 (approximately 30 feet bgs). The geometry, as specified in the FS groundwater model, differs slightly from the feature shown on Figure 6-10 to fit the resolution of the groundwater model grid.
  - The impermeable cap was applied to Site uplands, but excluded the 100-foot-wide habitat area along the shoreline (shown on Figure 6-1 of the FS main text).

- **Alternative 6.** Alternative 6 includes an impermeable cap, funnel and gate, solidification of soil containing 2 or more feet (cumulative thickness) of DNAPL to a maximum depth of 20 feet bgs, solidification of deep DNAPL-impacted soil in the RR DNAPL Area and in the vicinity of MC-1, and excavation and removal of DNAPL-impacted soil in the Quendall Pond Upland (QP-U) DNAPL Area.

  - The footprint of the QP-U DNAPL Area excavation in the groundwater model follows the design shown for Alternative 6 on Figure 6-12 of the FS main text and extends approximately 19 feet deep. The geometry is slightly different from the design to fit the groundwater model grid. Backfill is assumed to be gravel with relatively high hydraulic conductivity (1.0×10^{-2} cm/sec).
  - All groundwater model cells within the shallow DNAPL zones shown on Figure 6-12 of the FS main text were solidified to a depth of approximately 20 feet bgs and groundwater model cells within deep DNAPL zones were solidified to 2 feet below the DNAPL-impacted soil. In addition, a one-cell buffer (approximately 25 feet) around all the treated zones was solidified.
  - The funnel and gate design shown on Figure 6-12 was replicated in the groundwater model in model layers 2 (ground surface) through 8 (approximately 30 feet bgs). The geometry, as specified in the FS groundwater model, differs slightly from the feature shown on Figure 6-12 to fit the resolution of the groundwater model grid.
  - The impermeable cap was applied to Site uplands, but excluded the 100-foot-wide habitat area along the shoreline (shown on Figure 6-1 of the FS main text).

- **Alternative 7.** Alternative 7 includes solidification of all upland DNAPL-impacted soil and the impermeable upland cap featured in previous alternatives.

  - All groundwater model cells representing a hydrocarbon-source zone (Figure A-2 and A-3) were assumed to be solidified. In addition, a one-cell buffer (approximately 25 feet) around the zones and an approximate 2-foot-thick layer below the source was solidified.
  - The impermeable cap was applied to Site uplands, but excluded the 100-foot-wide habitat area along the shoreline. Recharge over solidified soil outside of the cap was also set to zero.

- **Alternative 8.** Alternative 8 features excavation of all upland DNAPL-impacted soil and the installation of the funnel and gate and the impermeable upland cap.
  - All groundwater model cells representing hydrocarbon source areas (as depicted on Figure A-2 and A-3) were excavated. The backfill in Alternative 8 was assumed to be excavated soil that is treated and reused as backfill. Backfill is assumed to have a relatively low hydraulic conductivity (1.0×10^{-4} cm/sec).
  - The funnel and gate was simulated in the groundwater model from model layer 2 (fill) through 8 (approximately 30 feet bgs). The groundwater model assumes a funnel and gate treatment wall but subsequently, the wall was removed from the alternative because it did not add significant benefit.
  - The impermeable cap was applied to Site uplands, but excluded the 100-foot-wide habitat area along the shoreline.

- **Alternative 9.** Approximately the upper 15 feet of the Shallow Alluvium within the area of MCL exceedances is excavated in Alternative 9. This alternative also includes solidification of DNAPL-impacted soil below 15 feet bgs, and the upland cap.
  - Groundwater model cells representing hydrocarbon-source zones (as depicted on Figure A-2 and A-3) that are more than approximately 15 feet bgs[9] were assumed to be solidified, including a one-cell buffer around the zones and an approximate 2-foot-thick cell below the hydrocarbon source. Groundwater model cells within the Site uplands that were shallower than approximately 15 feet bgs were assumed to be excavated and backfilled with low hydraulic conductivity treated soil (1.0×10^{-4} cm/sec).
  - Similar to previous alternatives, the cap was applied to the Site uplands, excluding the 100-foot-wide habitat area along the shoreline (shown on Figure 6-1 of the FS main text).

- **Alternative 10.** Alternative 10 features excavation of all Shallow Alluvium soils within the area of MCL exceedances and the installation of the impermeable upland cap and the pump and treat system.
  - Only benzene, naphthalene, and arsenic are simulated with the groundwater model for Alternative 10. The alternative is designed to completely remove benzo(a)pyrene source material and the groundwater model assumes no contaminated residuals. Therefore, the groundwater model prediction should be that benzo(a)pyrene would restore immediately. However, the modeled pre-remediation extent of benzo(a)pyrene is greater than the modeled extent of soil removal; therefore, the groundwater model (if run for benzo[a]pyrene) would still predict exceedances.

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[9] Based on resolution in cell grid; actual depth ranges from 13 to 27 feet bgs, with an average of 15 feet bgs.

- The entire Shallow Alluvium within Site uplands within the area of the benzo(a)pyrene and arsenic plumes is assumed to be excavated in Alternative 10. Low hydraulic conductivity treated soil (1.0×10^{-4} cm/sec) is used to backfill the excavation.
- The pump and treat system was simulated in the groundwater model as described in the Section A5.2.
- Similar to previous alternatives, the cap was applied to the Site uplands, excluding the 100-foot wide habitat area along the shoreline (shown on Figure 6-1 of the FS main text).

A3.6 Remedial Alternatives Groundwater Model Results

Empirical Site data were used to estimate flow and contaminant transport parameters and source concentrations used in the groundwater model to best represent pre- and post-remedy concentrations for the remedial alternatives. As previously indicated, the groundwater model incorporates simplifying assumptions to provide an approximate representation of complex Site conditions. These simplifying assumptions introduce inherent uncertainty in the groundwater model results. To address the uncertainty, the groundwater modeling assumptions are consistently applied in evaluating the range of alternatives. Further, the groundwater model results are evaluated in relative versus absolute terms. By evaluating a result on a large diffuse scale such as plume volume and, more importantly, comparing the relative change in the groundwater model results, much of the uncertainty associated with absolute predictions by the contaminant transport model is mitigated. Therefore, the results presented below should be interpreted within a comparative analysis of the relative benefit from each alternative.

Groundwater model results for the evaluation of the remedial alternatives are presented in Tables A-6 and A-7. The extent of groundwater contamination predicted by the groundwater model is illustrated as plume extent in plan view on Figures A-13 through A-17, and in cross section on Figures A-18 through A-21. The contaminant transport model results were calculated at 3-year time intervals to assess restoration timeframe over the entire 100 year simulation period. The groundwater model output at time 100 years was processed to produce different metrics to compare the individual remedial alternatives after 100 years of implementation. These metrics include the following:

- **Plume Volume.** The aggregate plume volume is defined as the volume of groundwater that exceeds the PRG of one or more of the primary Site COCs (5 µg/L for benzene, 0.2 µg/L for benzo(a)pyrene, 1.4 µg/L for naphthalene, and 10 µg/L for arsenic). The volume was calculated from the groundwater model output by summing the volume of cells (17 ft^3 to 33,000 ft^3 per cell; 4,400 ft^3 on average) whose concentration exceeded one or more of the PRGs, and then multiplying the sum by the effective porosity (0.25). In alternatives that include solidification (Alternatives 3, 5, 6, 7, and 9), only the volume outside of solidified soil is included in the calculation. Plume volumes are presented in Table A-6 for the Shallow Aquifer and Deep Aquifer combined, and for the upland Deep Aquifer only. Volumes are reported in units of millions of gallons of groundwater.

Plume volumes for each of the primary Site COCs for the Shallow Aquifer and Deep Aquifer combined were also calculated and are presented in Table A-7. The groundwater model results indicate arsenic plumes for Alternatives 1 through 9 that are larger than the pre-remediation plume. The expansion of these plumes is the result of not using modeled source propagation to define the initial conditions for arsenic. The increase in arsenic plume volume is due to the groundwater model adjusting the assigned concentrations to establish a new equilibrium across groundwater model cells based on concentration gradient. More discussion of the use of plume propagation to produce initial conditions and the implications are presented in Section A3.2.

- **Mass Flux:** The Mass Flux for each primary Site COC was calculated for each of the alternatives at the groundwater model boundary representing the lakebed sediments. These are not estimates of the total mass flux to Lake Washington because they do not include sediment processes or offshore DNAPL. Rather, the results were used to compare the relative reduction in mass flux into the lakebed sediments for each alternative. The mass flux results generated by the groundwater model were used to only provide a relative comparison between remedial alternatives and were not used as inputs to the Reible sediment transport model discussed in Appendix B. For that model, empirical sediment porewater data were used.
- **Dissolved Plume Contaminant Mass.** Dissolved plume contaminant mass was calculated for each of the primary Site COCs under each remedial alternative. Dissolved mass was calculated by summing the products of COC concentration, porosity, and volume of model cells within each plume. In alternatives that include solidification (Alternatives 3, 5, 6, 7, and 9), only the mass outside of solidified soil was included in the calculation. These results are presented in Table A-7.
- **Restoration Timeframe.** The restoration timeframe of each of the primary Site COCs was estimated as the time in years when predicted concentrations in every groundwater model grid cell were below their respective PRG as presented in Table A-6. The groundwater model results indicate that none of the remedial alternatives achieves groundwater restoration (defined as concentrations for each of the four primary Site COCs below their respective PRGs) for all of the Site COCs. However, Alternatives 8 and 10 achieve restoration of benzene and naphthalene. Alternative 10 would achieve restoration of benzo(a)pyrene before the end of the model run (100 years), but this restoration is based on the unrealistic assumption that the entire source of benzo(a)pyrene is removed and there are no excavation or dredging residuals. A sensitivity analysis (see Section A5.1) indicates that residuals would cause benzo(a)pyrene MCLs to be exceeded for more than 100 years.
- **Relation to University of Texas (UT) Model.** Groundwater discharge fluxes were also calculated to evaluate seepage rate reduction associated with upland capping and funnel and gate technologies, to support sediment modeling presented in Appendix B3 of this FS. Groundwater discharge flux for offshore and nearshore areas are tabulated in Table A-8.

A3.7 Sensitivity Analysis of FS Groundwater Model Results

Contaminant fate and transport input parameters for the FS groundwater model were based on site-specific data, literature values, and best professional judgment as discussed in Section A3.2. A sensitivity analysis was conducted to assess relative uncertainty in the FS groundwater model results attributable to contaminant fate and transport parameter assumptions. Using Alternatives 1, 7, and 8, a sensitivity analysis of the sorption coefficient (K_d), half-life, and source area concentration was conducted. The parameters were varied one at a time while the two remaining values were held at base value. The values used in the sensitivity analysis are described below and are presented in Table A-9:

- **K_d.** Five hundred percent (five times) of the base K_d was used as the high K_d value. Twenty percent (one fifth) of the base value was used for the low value for symmetry.
- **Half-Life.** Arsenic does not decay and, therefore, was not included in the sensitivity analysis. Half-life ranges for hydrocarbons were set as follows:
  - **Benzene.** The FS groundwater model uses the mid-range anaerobic half-life for benzene of 720 days (see Section A3.1.2) as a base value. In the sensitivity analysis, the lowest anaerobic half-life reported in Howard et al. 1991 was used for the low half-life value, and a value of five times the base half-life (3,600 days) within the reported range of half-lives estimated from field studies (220 days to no degredation: Aronson 1997) was used for the high half-life value.
  - **Naphthalene.** Because only one anaerobic half-life was reported for naphthalene (Howard et al. 1991), the naphthalene low half-life was reduced from the base value by the same proportion as for benzene. The high half-life value was taken as 500 percent of the base value.
  - **Benzo(a)pyrene.** The low anaerobic half-life was set to 1,484 days, the lowest anaerobic half-life reported for chrysene in Howard et al.1991. Benzo(a)pyrene decay was not simulated in the high half-life sensitivity run.
- **Source Area Concentration.** The high and low source area concentrations were 150 and 50 percent, respectively, of the base value used in the groundwater model. This base value was calculated from the mean of detected concentrations within the source areas, and the high and low values fall within the range of detected values.

A3.7.1 Sensitivity Analysis Results

The sensitivity analysis results for individual COCs are presented in Table A-10 and sensitivity analysis results as aggregate plume volume are presented in Table A-11. The aggregate results were reduced to produce the maximum variation from the base results to produce Figure A-22. The brackets on Figure A-22 reflect the sensitivity results that were maximum departures from the base groundwater model results (worst case and best case). Best case results are from the parameter set that produced the smallest value and worst case results are results from the parameter set that produced the largest value. The bars on Figures A-22 represent the base case groundwater model result presented in Section A3.6. In addition, Figure A-22 compares aggregate plumes of groundwater that exceed PRGs, as well as aggregate plumes of groundwater that exceed only MCLs (5 µg/L for benzene, 0.2 µg/L for benzo(a)pyrene, and 10 µg/L for arsenic). The aggregate plumes that exceed only MCLs do not include naphthalene, which has no MCL.

Sensitivity analysis was only performed on Alternatives 1, 7, and 8. The variability in aggregate plume volume groundwater model results of the remaining alternatives were estimated by a linear interpolation and extrapolation of the sensitivity analysis results from Alternatives 1, 7, and 8. Linear regression of sensitivity analysis-derived best and worst case volumes (when compared to base case groundwater model results) were generated for Alternatives 1, 7, and 8 and those regressions are shown on Figure A-23. The resulting regressions were then used to estimate best case and worst case aggregate plume volumes for the remaining alternatives. For example, the estimated best case value for an alternative is estimated as the y value of a point that falls on the best case regression line and has an x value equal to that alternative's base case result. Figure A-23 shows groundwater model results generally fit close to their regression lines and have a minimum R-squared value of 0.992; therefore, the linear approximation provides a reasonable estimate of sensitivity results for the remaining alternatives' best case and worst case. Results estimated by interpolation and extrapolation are shown for the aggregate plume exceeding PRGs on Figure A-24 and for the aggregate plume exceeding only MCLs (excludes naphthalene) on Figure A-25. The bars on Figures A-24 and A-25 represent the base case groundwater model result as described above in Section A3.6 and the brackets for Alternatives 2, 3, 4, 5, 6, 9, and 10 represent the variation estimated from the alternatives base result and the linear regressions presented on Figure A-22. Table A-12 presents both the variation in aggregate plume volume derived from sensitivity analysis (Alternatives 1, 7, and 8) and the linear regression-derived variation (estimated).

Sensitivity analysis results by COC were treated similarly to aggregate plume results. Sensitivity analysis results of plume volume by COC are shown on Figure A-26, their linear regression is shown on Figure A-27 and the sensitivity analysis-derived and linear regression-derived variation in plume volume by COC are displayed on Figure A-28 and in Table A-13. Similarly, sensitivity analysis results of plume mass by COC are shown on Figure A-29, their linear regression is shown on Figure A-30, and the sensitivity analysis-derived and linear regression-derived variation in plume mass by COC are displayed on Figure A-31 and in Table A-14. Lastly, similar to previously discussed metrics, sensitivity analysis results of mass flux by COC are shown on Figure A-32, their linear regression is shown on Figure A-33, and the sensitivity analysis-derived and linear regression-derived variation in mass flux by COC are displayed on Figure A-34 and in Table A-15. See Section A3.6 for a definition of plume volume, plume mass, and mass flux.

A4 Excavation Dewatering Analysis

The FS groundwater model was used to evaluate pumping rates required to achieve excavation dewatering criteria for Site remedial alternatives. To effectively remove and handle contaminated soil and to maintain excavation stability, dewatering would be required during soil excavation to meet two goals:

1. Dewater the contaminated soil located below the water table such that excavation occurs either in unsaturated (dry) conditions or the water level is lowered enough to allow installation of shoring; and
2. Depressurize the Deep Aquifer to prevent destabilization of the excavation bottom. The Deep Aquifer is a semi-confined aquifer with a potentiometric surface (head) 20 to 40 feet above the bottom of the Shallow Alluvium.

The FS groundwater model was used to estimate dewatering rates of excavations (for soil removal and DNAPL collection trench installation) in Alternatives 3 4, 6, and 8. Dewatering required for Alternative 9 and 10 was estimated with groundwater modeling completed early in the FS process, which is presented in Section A5.

The following sections discuss three topics: structural and boundary condition modifications to the RI groundwater model to develop the groundwater model used to evaluate FS dewatering criteria (Section A4.1), the constructability assumptions that determine dewatering criteria (Section A4.2), and dewatering groundwater model results (Section A4.3).

A4.1 Modifications to Develop Dewatering Groundwater Model

The following modifications were made to the groundwater flow component of the RI groundwater model for the FS dewatering evaluation. These modifications include both structural modifications and the addition of boundary conditions, such as the following:

- The addition of four to five layers in the Shallow and Deep Aquifers to improve the vertical resolution of excavation boundary conditions (i.e., sheet pile walls and dewatering wells).
- The addition of sheet pile walls simulated with MODFLOW's HFB Package. HFB boundary conditions were inserted between groundwater model cells around the perimeter of the excavation cell and extend from model layer 1 to the approximate sheet pile embedment depth reported in Tables A-16 and A-17**.** The HFB boundary conditions were given a small conductance value (1 x 10^{-20} cm^2/sec) to make them effectively impermeable.
- Dewatering wells were inserted in the groundwater model using the multi-node well package (Halford and Hanson 2002). Wells were placed within the sheet pile wall enclosures**.** Wells were screened in the top 10 to 15 feet of the Deep Aquifer, with the top of the screens being at the interface of the Shallow and Deep

Aquifers. The hydraulic conductivity of cells within excavation cells from groundwater model layer 2 to the approximate excavation depth listed in Tables A-16 and A-17 were given a large value (1.0 x 10^9 feet/day) to simulate an open excavation.

- Recharge was reduced to 0 inches/year within the excavation.

A4.2 Constructability Assumptions

Dewatering criteria are dependent on constructability assumptions. Excavations can either be done in the wet or in the dry. The minimum drawdown required for dry excavations is prescribed by the maximum depth of the excavation; in the case of wet excavations, minimum drawdown is determined by constructability requirements for installation of tieback anchors in the shoring walls.

Maximum excavation depths are presented in Tables A-16 and A-17. Calculations and assumptions used to estimate constructability requirements are detailed in Appendix F of this FS and the requirements are as follows:

- Tieback anchors for shoring walls are not required for excavations shallower than 16 feet and, therefore, do not require depressurization if done in the wet;
- Excavations between 16 and 22 feet deep require a minimum depth to water of 8 feet bgs to accommodate construction of tieback anchors;
- Excavations between 22 and 27 feet deep require a minimum depth to water of 13 feet bgs to accommodate construction of tieback anchors; and
- Excavation between 27 and 34 feet require a minimum depth to water of 19 feet bgs to accommodate construction of tieback anchors.

In addition to dewatering requirements in the Shallow Aquifer, the Deep Aquifer must also be depressurized to prevent destabilization of the excavation floor. For the purposes of this analysis, it was assumed that the confined head at the top of the Deep Aquifer must be below the minimum elevation of the excavation floor for a dry excavation. In wet excavations, the head in the top of the Deep Aquifer must be at or below the elevation of the static water level within the excavation. The maximum excavation depths (minimum excavation elevation) and constructability requirements were used to determine the dewatering criteria targets for pumping optimization using the dewatering groundwater model.

A4.3 Dewatering Groundwater Modeling Approach

Dewatering and depressurization flow rates were estimated using the groundwater model in an iterative process in which pumping rates and the number of wells were adjusted until dewatering criteria were achieved under steady state conditions.

Dewatering and/or depressurization flow rates were estimated for each of the cells shown on Figure 6-17 of the FS main text (Alternative 8), for the Quendall Pond cell depicted on Figure 6-6 of the FS main text (Alternatives 4 and 6), and for the DNAPL collection trench depicted on Figure 6-4 of the FS main text (Alternatives 3 and 4).

A4.4 Dewatering Groundwater Modeling Results

Groundwater model results for the dewatering evaluation for wet excavations are presented in Table A-16. Similarly, results for dry excavation dewatering are presented in Table A-17. Because of the confined head in the Deep Aquifer, it is estimated that excavations requiring dewatering of the Shallow Aquifer would also require depressurization of the Deep Aquifer.

A5 Additional Evaluations for Alternative 9 and 10

This section describes groundwater modeling done early in the FS process (Early FS groundwater model) for the following purposes:

- To perform additional sensitivity analysis on the effect of two parameters on groundwater-model-predicted restoration timeframe for Alternative 10: 1) the presence of heterogeneities in the Deeper Alluvium and 2) the presence of contaminated residuals after excavation;
- To develop conceptual design criteria, including optimal well locations and flow rate, of a pump and treat system used in Alternative 10; and
- To estimate construction dewatering flow rates needed to facilitate removal of contaminated materials as part of Alternatives 9 and 10.

Similar to the evaluations presented in previous sections, this evaluation uses a refined version of the groundwater flow and contaminant transport model documented in Appendix D of the RI (Anchor QEA and Aspect 2012). The Early FS groundwater model described in this section features the same flow and transport parameters as the FS groundwater model documented in Sections A3 and A4 of this appendix, but has the following differences:

- Concentrations specified for the DNAPL boundary conditions for the Early FS groundwater model were based on data provided in the draft RI Report, while the concentrations for the FS groundwater model were based on data provided in the final RI Report. Differences were as follows:
  - For benzene, 2,800 µg/L was used in all DNAPL zones that were a source of benzene in the Early FS groundwater model, rather than zone-specific concentrations noted on Figure A-3 (ranging between 200 and 12,000 µg/L);
  - For naphthalene, 16,000 µg/L was used in the Early FS groundwater model rather than 11,000 µg/L shown on Figure A-3;
  - For benzo(a)pyrene, 20 µg/L was used in the Early FS groundwater model rather than 130 µg/L shown on Figure A-3; and
  - For arsenic, 53 µg/L was used in the Early FS groundwater model rather than 39 µg/L.
- Zone 3 depicted on Figure A-1 was included as a source in the Early FS groundwater model.
- The Early FS groundwater model included 11 model layers rather than 20 in the FS groundwater model. The Early FS groundwater model includes the 10 layers that comprise the RI model and one additional 2-foot-thick layer located at the top of the Deep Aquifer, used to simulate the DNAPL present at the top of the Deep Aquifer near the Railroad Area. Additional layers were added for the simulation of aquifer heterogeneity as described in Section A5.1.1.

- Transport model results for the Early FS groundwater model were printed at a resolution of up to 15 years rather than 3 years.

These differences are not expected to significantly alter the results or conclusions of the analyses described in this section.

The sensitivity analysis is described in Section A5.1. The optimization of the pump and treat system is documented in Section A5.2. The dewatering evaluation used to support cost estimates for the implementation of Alternatives 9 and 10 are documented in Section A5.3.

A5.1 Restoration Timeframe Sensitivity Analysis

The groundwater model was used to simulate the restoration timeframe following the assumed removal of sources from the Shallow Alluvium and DNAPL from the Deeper Alluvium (Alternative 10). The steps to setup and run the groundwater model to estimate the restoration timeframe were the same as for the FS groundwater model, except that 200-year restoration periods (in addition to 100-year restoration periods) were also conducted for selected conditions when MCLs were not achieved within 100 years.

The effect of varying groundwater model input assumptions on groundwater model results (i.e., sensitivity analyses) was evaluated to assess the range of uncertainty in the groundwater model predictions. Model input parameters evaluated in the sensitivity analyses included the following:

- **Aquifer heterogeneity.** The FS groundwater model assumes the Deeper Alluvium is homogeneous. However, based on Site boring logs, some areas of the upper portion of the Deeper Alluvium exhibits heterogeneous conditions, including low-permeability lenses of silt and silty sand within a matrix of more uniform sand and gravel. Some portions of the Deep Aquifer, particularly at greater depths, exhibit more homogeneous characteristics and do not appear to contain low-permeability layers of silt or silty sand.
- **Presence of excavation residuals.** The FS groundwater model assumes no residual contamination left behind after removal actions, which is deemed to be highly unlikely due to the complexity of the Site.

The following sections describe groundwater model modifications to evaluate aquifer heterogeneity (Section A5.1.1) and groundwater model modifications to evaluate excavation residuals (Section A5.1.2).

A5.1.1 Aquifer Heterogeneity

A common approach for constructing larger-scale groundwater models is to use an equivalent porous media approach to define aquifer parameters. This approach assumes that, on a site-wide scale, changes in groundwater velocities from smaller-scale aquifer heterogeneities are represented by averaging aquifer parameters (i.e., hydraulic conductivity) resulting in an average flux. However, this assumption is often not appropriate when simulating contaminant transport or evaluating individual chemical transport processes on a smaller scale (Zheng et al. 1995).

The Deeper Alluvium is predominantly sand and gravel but silty sand lenses and silt lenses are also present. For example, borings BH-5B, BH-21B, and SWB-3 contain intervals of silty sand between 1- and 9-feet thick near the top of the Deeper Alluvium, and borings BH-5B and BH-30C have a 0.5-foot thick silt lens from 45 to 50 feet bgs. Based on a review of the boring logs, two representative lower-permeability lenses within the Deeper Alluvium were incorporated into the groundwater flow model: a silty sand layer, 5-feet thick, approximately 45 feet bgs; and a silt layer 0.5-foot thick at 50 feet bgs. A summary of the boring log analysis is presented in Table A-18.

A sensitivity analysis to evaluate the potential impact of fine-grained layers in the Deeper Alluvium on groundwater model results was conducted using the Early FS groundwater model. Site heterogeneity was evaluated using the following modification to the groundwater model:

- A finer-grained layer was placed in the middle of the Site as a representative case. In actuality, low-permeability layers were observed within the upper portions of the Deeper Alluvium at multiple locations across the Site, including on the eastern (e.g., BH-30C) and western (e.g., BH-20C) areas of the Site. The full distribution of all fine grain layers throughout the Deeper Alluvium is unknown; therefore, this evaluation was completed at the scale of the single representative fine-grained layer placed within the groundwater model. Results must be interpreted while considering that this is one of many fine-grained layers present in the groundwater model. Lower-permeability zones were placed within the site-wide groundwater model so that groundwater flow within the zones and interaction between the fine-grained zones and surrounding sand and gravel were calculated by the groundwater model.
- Horizontal hydraulic conductivity for the finer-grained units was estimated from Table 2.2 of Freeze and Cherry (1979) at 1×10^{-4} cm/s and 1×10^{-6} cm/s for the silty sand and silt, respectively. Anisotropy (ratio of horizontal to vertical hydraulic conductivity) was assumed to be the same as for the rest of the Deeper Alluvium (40:1). The silty sand zone was assumed over an area of 80 by 85 feet and the silt zone was assumed over an area of 40 by 45 feet; both are longer in the direction of groundwater flow. Based on the small area of the zones relative to the groundwater model grid spacing, the grid spacing was telescoped (refined) to 5 feet. To better resolve vertical flow paths, 15 additional layers were also added to the groundwater model grid.
- Dispersivity was reduced within fine-grained zones to simulate dispersion over a shorter flow path length (versus site-wide transport). Longitudinal dispersivity within the fine-grained zones is assumed to be 0.5-foot, and transverse and vertical dispersivity are assumed to be 0.05-foot and 0.005-foot, respectively. Initial concentrations within the finer-grained zones were specified at 8,400 µg/L for benzene (as measured at BH-20B, one of the locations where finer-grained layers have been observed), 6,400 µg/L for naphthalene, 20 µg/L for benzo(a)pyrene, and 53 µg/L for arsenic.

The entire groundwater model domain was used for this analysis and initial conditions outside of the fine-grained layers remained unchanged from the baseline simulation. Since this evaluation focuses on the scale of a single representative fine-grained layer, additional virtual observation wells were added to the groundwater model cells within the fine-grain zones with the highest concentration after the groundwater model simulation, or in the cells where COC concentrations remained above the MCLs the longest during the groundwater model simulations.

Restoration timeframes were estimated for three pumping scenarios: no pumping, pumping at the optimized pumping rate (90 gpm: see Section A5.2), and pumping at twice the optimized pumping rate. Restoration timeframes calculated under these scenarios assuming a homogeneous aquifer or a heterogeneous aquifer are presented in Table A-19. If restoration for a COC is not achieved within the timeframe of the groundwater model (100 or 200 years), the highest remaining concentration of that COC is provided. In this analysis concentrations were compared to the following cleanup levels: 1.4 µg/l for naphthalene, 5 µg/L for benzene, 0.2µg/L for benzo(a)pyrene, and 10 µg/L for arsenic. Results were as follows:

- When Deeper Alluvium heterogeneity is simulated within the natural flushing (i.e., no pumping) scenario, benzene attenuates to concentrations below the MCL within 30 years. Arsenic and benzo(a) pyrene still exceed their respective MCLs after 100 years. The highest residual arsenic concentration is 53 µg/L and the highest residual benzo(a)pyrene concentration is 20 µg/L, both located within low-permeability layers of the Deeper Alluvium.
- Under the homogeneous natural flushing assumption, benzene in the Deeper Alluvium attenuates to concentrations below the MCL of 5 µg/L within 13 years. Naphthalene attenuates below the PRG (1.4 µg/L) within 53 years. Groundwater-modeled predicted concentrations of arsenic and benzo(a)pyrene in groundwater exceed their respect MCLs after 200 years. The highest residual arsenic concentration in the Deeper Alluvium is 33 µg/L (MCL equal to 10 µg/L) and the highest residual benzo(a)pyrene concentration is 4.2 µg/L (MCL equal to 0.2 µg/L).

Pump and treat results in a slight improvement (reduction) in the restoration timeframes under both heterogeneous and homogeneous assumptions. Doubling the optimized extraction flowrate (based on plume capture) did not significantly improve restoration timeframe under either heterogeneous or homogeneous assumptions.

A5.1.2 Excavation Residuals Sensitivity Analysis

Contaminant removal by excavation could leave behind residual contamination at the base of the excavation. This section evaluates the potential for such residuals to extend the restoration timeframe.

The potential contribution from residual contamination was evaluated by inserting a 2-inch layer of contaminated Shallow Alluvium soil at the base of the Shallow Alluvium, representing residual benzene and benzo(a)pyrene. In total, seven additional layers were added to the groundwater model to allow simulation of contaminant transport at a higher resolution. These seven included the approximately 2-inch layer and six layers below it. The initial conditions applied to the groundwater model assumed sorbed concentrations of 5 milligrams/kilograms (mg/kg) (benzene) and 10 mg/kg (benzo[a]pyrene) within the 2-inch layer throughout the Site. The initial dissolved concentrations were calculated assuming soil:water equilibrium by applying their respective K_d values. The groundwater model simulates 100 years of transport following the excavation and assumes a natural gradient and a homogeneous Deeper Alluvium. Initial COC concentrations in the Deeper Alluvium were set to zero to estimate the contribution from the residual layer.

Including the residual contamination layer in the groundwater model run did not increase the time (13 years) for benzene to attenuate below the MCL relative to that estimated by the natural flushing simulation. The residual layer contributed to a maximum additional benzo(a)pyrene concentration of 1.3 µg/L after 100 years. The estimated volume of groundwater exceeding the benzo(a)pyrene MCL was 14 million gallons. This value was used for the error bar shown on Figure A-28 for benzo(a)pyrene plume volume under Alternative 10.

A5.2 Pump and Treat System Optimization

Pumping wells were introduced to the groundwater model to evaluate the effect of pump and treat on the restoration timeframe. The groundwater model was first used to optimize extraction well placement and pumping rate so as to achieve complete plume capture (described below). The new groundwater flow field for the pumping condition was then imported into the contaminant fate and transport model to predict contaminant elution and, as a result, restoration timeframe.

Pumping wells are simulated within the MODFLOW groundwater model using the multi-node well package (Halford and Hanson 2002). The Multi-node well package simulates pumping across multiple MODFLOW layers and calculates drawdown within the well. The package takes into account the head, hydraulic conductivity and grid spacing of pumping cells, and represents the pumping impacts across multiple layers within the groundwater model.

The number, location, and flow rate of groundwater pumping wells was adjusted under steady state conditions to optimize hydraulic capture while reducing total volume extracted. Each pumping well was screened in the top of the Deeper Alluvium, approximately 30 to 50 feet bgs, to optimize capture of contaminated groundwater.

MODFLOW's particle tracking package, MODPATH (Pollack 1994), was used to evaluate the effectiveness of capture. MODPATH results show the advective movement of particles as flow lines through an established groundwater flow field. Three lines of 100 particles (elements used to designate flow lines) representing the extent of the arsenic, benzene, and benzo(a)pyrene plumes were placed across the Site, approximately 10 feet below the top of the Deeper Alluvium. The particles were then traced forward through the groundwater model to represent the capture area. As the flow rate increased, the width of capture also increased. Complete groundwater capture is achieved when all flow lines from the plume edges are captured by the wells.

Particle tracks representing capture predicted by the groundwater model are presented on Figures A-35 and A-36. Based on the groundwater modeling, steady-state hydraulic capture is achieved with a minimum of six wells and a total flow rate of 90 gpm distributed evenly between the wells (15 gpm/well). This configuration was implemented in the contaminant transport model. Capture was also achieved by a flow rate of 80 gpm from 12 wells. The 90 gpm scenario was chosen because it would require less infrastructure and, therefore, lower capital costs.

A5.3 Construction Dewatering - Alternatives 9 and 10

To effectively remove and handle contaminated soil and to maintain excavation stability, dewatering would be required during soil excavation to meet two goals: 1) dewater the saturated contaminated soil in place such that excavation occurs in unsaturated conditions and 2) depressurize the Deeper Alluvium to prevent heaving or destabilization of the excavation bottom. The Deeper Alluvium is a semi-confined aquifer with a potentiometric surface (head) 20 to 40 feet above the bottom of the Shallow Alluvium.

A5.3.1 Excavation Dewatering (Shallow Alluvium)

Means and methods for dewatering the Shallow Alluvium would be determined during remedial design but may include temporary sumps within the open excavation and/or well points outside the excavation. Sumps are an effective means of dewatering excavations within lower permeability material where the groundwater heads need only to be depressed several feet. If sumps are inadequate for dewatering, closely-spaced vacuum well points outside the excavation footprint would be required.

A5.3.2 Depressurization of Deeper Alluvium

Reduction of head in the Deeper Alluvium would require pumping wells screened across the Deeper Alluvium. Pumping wells have the ability to effectively dewater large areas in permeable sediments and may produce large amounts of water. Dewatering pumping wells typically consist of 6- to 12-inch casings installed in 10- to 16-inch boreholes. Screen designs and filter packs are specified based on the grain size of the water-bearing zone. Submersible pumps are generally placed inside the well casing near the bottom of the screened interval.

To limit the potential for contaminant carry down, depressurization wells would be completed using double casing drilling techniques (sealing off the Shallow Alluvium prior to advancing drilling through the Shallow Alluvium and into the Deeper Alluvium) similar to that done during installation of wells BH-30C and BH-20C.

A5.3.3 Estimated Excavation Dewatering Flow Rates (Shallow Alluvium)

An analytical solution was used to estimate dewatering required for implementation of Alternative 9. The volume of water required to effectively dewater an excavation within the Shallow Alluvium is directly proportional to the average hydraulic conductivity of the Shallow Alluvium and increases the closer the excavation is to Lake Washington.

For open excavations (i.e., no groundwater cutoff), preliminary volumes for dewatering were first estimated analytically by assuming an equivalent well radius (Powers 1992) equal to that of an expected excavation cell size ranging from 0.1- to 1-acre to an average depth of 20 feet bgs[10]. Assuming the hydraulic conductivities and excavation heads from the calibrated groundwater model, we estimate that 60 to 100 gpm would flow into an excavation near the Railroad Area (BH-30) under steady-state conditions. Flow rates would increase with decreasing distance to Lake Washington. Near the shoreline (e.g.,

[10] As noted, the estimated dewatering flowrate was based on an assumed average excavation depth of 20 feet. Alternative 9 assumes an average excavation depth of 15 feet; therefore, this evaluation is considered conservative.

near BH-20), estimated flow rates range from 300 to greater than 1,000 gpm for cell sizes ranging from 0.1- to 1-acre, respectively.

The calculation assumes steady state conditions, whereas initial flow rates would be greater to reduce aquifer storage. The estimate does not account for surface runoff potentially entering the excavation.

If sheet piles or other methods are used to isolate excavation cells and limit lateral leakage from the Shallow Alluvium, seepage would occur through the bottom of the excavation. Assuming an average excavation depth of 20 feet, an average of 15 feet below the water table, approximately 1 to 56 gpm would enter an excavation cell of 0.1 to 4 acres, respectively.

A5.3.4 Estimated Depressurization Flow Rates (Deeper Alluvium)

Depressurization of the Deep Aquifer would be required for excavations included in Alternative 10. Flow rates required to depressurize the Deeper Alluvium unit were calculated by a similar method assuming the head in the aquifer needs to be lowered to the same elevation as the excavation bottom, at an average depth of 35 feet bgs, for a net zero gradient across soils underlying the excavation. Assuming the hydraulic conductivities and excavation heads from the groundwater model, we estimate that several thousand gpm would need to be withdrawn from the Deeper Alluvium to achieve the necessary 32 feet of drawdown under steady-state conditions.

The higher hydraulic conductivity (3×10^{-2} cm/sec) of the Deeper Alluvium requires the higher flow rates to achieve depressurization; therefore, groundwater cutoff should be considered to reduce flow rates to achievable levels. Using the calibrated groundwater flow model, depressurization flow rates were predicted with assumed increasing sheet pile embedment. Sheet piles would be driven through the Shallow Alluvium, thereby cutting off shallow groundwater inflow to the excavation (which is also significant near shore). Because of the anisotropic nature of the Deeper Alluvium, increased sheet pile embedment into the Deeper Alluvium forces longer vertical groundwater flow paths and lower groundwater flow rates.

Estimated depressurization flow rates for the Railroad Area and shoreline are presented in Table A-20. They range from 52 to 740 gpm for an excavation cell size ranging from 0.25 to 2 acres with sheet piles driven 1.5 times the depth of the Shallow Alluvium and dewatering depth of 35 to 40 feet bgs. For similar size cells, the flow rates decrease to 100 to 400 gpm when the sheet pile wall is advanced 20 additional feet. The required flow rate to dewater a 2-acre area with sheet piles advanced to 1.5 times the Shallow Aquifer thickness plus an additional 40 feet is estimated to be 400 gpm; the estimate is 570 gpm when sheet piles are only advanced an additional 20 feet.

In all scenarios, the depressurization wells were placed inside the sheet pile wall and screened in the upper 20 feet of the Deeper Alluvium. An excavation encompassing the entire Site with a sheet pile embedment of approximately 80 feet bgs would require a dewatering rate of approximately 2,500 gpm as predicted by the groundwater model; however, at this large pumping rate, there are significant boundary affects, particularly at the upgradient constant head boundary, that lead to significant overestimation of required pumping.

The required number and location (spacing) of depressurization wells would be determined during remedial design, but preliminary groundwater modeling suggests a minimum of four wells arranged evenly within the interior of the sheet pile wall would be required to effectively dewater a 1-acre excavation located near the shoreline. The induced downward gradient along the outside of the sheet pile wall with the deepest embedment is 0.07 feet/foot. The depressurization radius of influence (defined as 0.5 feet of drawdown) would extend approximately 1,600 feet from the excavation.

A6 References for Appendix A

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Table A-1 Contaminant Fate and Transport Parameters[1]

Quendall Terminals
Renton, Washington

Hydrostratigraphic Unit	f_{oc} [2]	K_d [3] (L/kg) Benzene	Naphthalene	Benzo(a)pyrene	Arsenic[4]
Fill	0.09%	0.054	0.55	256	29
Shallow Alluvium	0.29%	0.18	1.8	856	29
Deep Alluvium	0.09%	0.054	0.55	256	29
Lake Sediments	0.29%	0.18	1.8	856	29

Contaminant	Half Life (days)[5]
Benzene	720
Naphthalene	258
Benzo(a)pyrene	4,000
Arsenic	Not Simulated

Notes:

Longitudinal dispersivity equals 7.5 feet, transverse dispersivity equals 1 foot, and vertical dispersivity equals 0.75 feet.

[1] Bulk density assumed constant at 1.7 g/cm^3 as in previous modeling studies (Retec 1998). Log K_{oc} assumed equal to 1.79 L/kg for benzene, 2.8 L/kg for naphthalene, and 5.47 for benzo(a)pyrene (Hart Crowser 1997 and Retec 1998).

[2] Referenced from Hart Crowser (1997) and Retec (1998).

[3] Soil/water sorption coefficient (K_d) = f_{oc} * K_{oc} .

[4] K_d for arsenic is from WAC 173-340-900, Table 747-3.

[5] Based on anaerobic half lives found in the literature and past modeling studies (Howard 1991, Aaronson 1997, and Retec 1998) as discussed in Section A3.1.2.

Abbreviations:

f_{oc} = fraction organic carbon
L/kg = liters per kilogram
K_d = sorption coefficient
K_{oc} = soil organic carbon-water partitioning coefficient
MTCA = Model Toxics Control Act

Table A-2 Measured Dissolved Oxygen Concentrations

Quendall Terminals
Renton, Washington

Monitoring Well	Temperature (°C)	Conductivity (μmhos/cm)	Dissolved Oxygen (mg/L)	ORP (mV)
BH-5A	14.69	348	0.62	213.5
BH-5B	15.06	445	1.14	-398.5
BH-18A	12.12	986	0.91	188.7
	15.21	900	0.9	53.9
BH-18B	13.1	309	0.5	235.9
	13.92	324	0.28	72.3
BH-19	11.67	877	1.34	227.2
	15.47	996	0.93	-80.8
BH-19B	12.51	406	0.51	229
	14.6	374	1.99	-384
BH-20A	13.23	467	0.45	203.2
	15.22	515	1.41	-378
BH-20B	13.12	450	0.26	204.8
	14.11	536	0.71	-52.9
BH-20C	16.09	153	1.44	-298
BH-21A	17.99	762	0.61	-51.4
BH-21B	12.8	512	0.56	196.1
	14.16	551	0.69	-96.5
BH-22	11.88	352	1.08	-13.2
BH-23	14.66	950	1.65	-322.1
BH-24	13.23	658	0.99	248.2
	13.86	773	0.4	-375
BH-25AR	17.4	731	0.35	-64.9
BH-26A	14.64	387	0.3	-3.7
BH-26B	12.99	604	0.24	-88
BH-28	13.17	490	0.91	220.8
	12.76	473	0.44	-67.8
BH-28B	13.41	353	1.18	230
BH-29A	15.6	482	0.2	-84.3
BH-29B	14.51	559	0.25	12.3
BH-30C	12.44	162	0.39	-433
RW-NS-1	14.15	1044	0.42	118.8
Average	14.04	554.00	0.77	-27.13

Notes:
Data referenced from Table C-3 of Appendix C of the Quendall RI (Anchor QEA and Aspect 2012).

Abbreviations:
°C = degrees Celsius
mg/L - milligram(s) per liter
mV = millivolts
ORP = oxidation-reduction potential
μmhos/cm = micro ohms per centimeter

Table A-3 Source Area Concentrations[1]

Quendall Terminals
Renton, Washington

DNAPL-Related COC Concentrations

Monitoring Well	Benzo(a)pyrene Concentration (µg/L)	Naphthalene Concentration (µg/L)	Benzene Concentration (µg/L)[2]				
			Zone 1	Zone 2	Zone 3	Zone 4	Zone 5
BH-5	362	16,000	-	-	-	-	31,000
BH-19	ND	25	-	-	-	-	59
BH-21A	24.6	2,100	4	-	-	-	-
BH-20A	11.7	10,000	-	-	-	-	7,900
BH-25A(R)	ND	11,000	-	510	-	-	-
BH-23	ND	300	-	-	-	350	-
RW-NS1	ND	760	-	-	-	58	-
RW-QP1	ND	11,000	-	-	-	-	7,700
Q9	Footnote 3	45,000	-	1,600	-	-	-
Q14-W	-	-	-	-	ND	-	-
Average	**133**	**11,000**	**4** [4]	**1,100**	**ND**[5]	**200**	**12,000**

Arsenic Concentrations

Monitoring Well	Arsenic Concentration (µg/L)
BH-19	25.3
BH-5A	53.8
BH-5	21.5
BH-25A(R)	13.5
BH-5B	10.3
BH-20B	50.9
BH-21B	109
BH-26B	31.8
BH-28B	34.2
Average	**39**

Notes:

[1] COC concentrations from RI Report Figures 5.2-1, 5.2-2, 5.2-8, 5.2-9, 5.2-14, 5.2-16 and 5.2-17 (Anchor QEA and Aspect 2012).

[2] Benzene DNAPL zones are shown on Figures A-1, A-2, and A-3.

[3] Excluded from average because value exceeded COC solubility.

[4] Not simulated in the model because of relative low concentration.

[5] Non-detect; therefore, not simulated in the groundwater model.

- Dash indicates well not located in hydrocarbon source zone.

Abbreviations:

COC = Chemicals of concern
ND = COC was not detected and therefore not included in average concentration value.
µg/L = micrograms per liter

Table A-4 Case Studies for Solidification of Coal Tar and Creosote Constituents

Quendall Terminals
Renton, Washington

Site Name	Site Location	Date	Hydraulic Conductivity	Source
South 8th Street Landfill Superfund Site	West Memphis, AR	1999-2000	$1x10^{-5}$ cm/sec	EPA 2009
Georgia Power Company - Manufactured Gas Plant	Columbus, GA	1992-1993	$1x10^{-5}$ cm/sec	EPA 1999; EPRI 2003
Wisconsin Fuel and Light - former MGP facility	Manitowoc, WI	1994-1995	$1.8x10^{-7}$ cm/sec	EPA 1999
J.H. Baxter - Renton Site	Renton, WA	2004	$1.x10^{-5}$ cm/sec	Wilk 2007; Hainsworth 2011
American Creosote Works	Jackson, TN	1999-2000	$1.x10^{-5}$ cm/sec	Wilk 2007; Hainsworth 2011

Abbreviations:
cm/sec = centimeters per second

Table A-5 Development of Remedial Alternatives

Quendall Terminals
Renton, Washington

Comparison of Backfill Materials

Backfill Type	Treatment Area	Volume of DNAPL Treated in Gallons[1]	Percent of Plume Remaining[2]		
			Benzene	Naphthalene	Benzo(a)pyrene
Treated Soil[3]	Area 1	29,281	68%	87%	97%
Imported Fill[4]	Area 1	29,281	68%	88%	96%

Comparison of Remedial Technologies and Treatment Areas

Remedial Technology	Treatment Areas	Volume of DNAPL Treated in Gallons[1]	Percent of Plume Remaining[2]		
			Benzene	Naphthalene	Benzo(a)pyrene
***In Situ* Solidification[5, 6]**	Area 1	29,281	66%	85%	99%
	Area 1 and 2	53,897	58%	82%	99%
	Area 1 through 3	87,422	52%	78%	99%
Excavation[3]	Area 1	29,281	71%	87%	97%
	Area 1 and 2	53,897	62%	83%	93%
	Area 1 through 3	87,422	53%	77%	87%
	Areas 1, 4, 5, and 6	145,480	61%	85%	83%

Notes:

[1] Volume calculation documented in Appendix E.

[2] Percent of pre-remediation plume volume remaining after 100 years after alternative implementation.

[3] Assumes excavation of DNAPL, on-site treatment, and backfill with treated soil ($K=1.0 \times 10^{-4}$ cm/s).

[4] Assumes excavation of DNAPL with off-site disposal and replacement with clean imported fill ($K= 1.0 \times 10^{-2}$ cm/s).

[5] Assumes *in situ* solidification of DNAPL.

[6] Percent plume remaining includes solidified zone.

Table A- 6 Evaluation of Remedial Alternatives - Aggregate Plume Volumes

Quendall Terminals
Renton, Washington

Alternative	Aggregate Plume Volume in MG[1]			
	All Aquifers Combined		Upland Deep Aquifer	
	Exceeds PRGs	Exceeds MCLs	Exceeds PRGs	Exceeds MCLs
Pre-Remediation	321	234	73.0	45.9
Alternative 1	323	241	73.5	46.6
Alternative 2	287	211	70.2	43.3
Alternative 3	233	162	57.0	32.7
Alternative 4	273	195	70.2	43.0
Alternative 5	224	155	57.3	32.8
Alternative 6	184	121	47.8	25.2
Alternative 7	65.0	51.7	23.3	16.0
Alternative 8	60.6	60.6	16.5	16.5
Alternative 9	74.4	53.3	26.0	16.2
Alternative 10	21.5	21.5	10.2	10.2

Alternative	Aggregate Plume Volume as Percent[1]			
	All Aquifers Combined		Upland Deep Aquifer	
	Exceeds PRGs	Exceeds MCLs	Exceeds PRGs	Exceeds MCLs
Alternative 2	89%	87%	96%	93%
Alternative 3	72%	67%	78%	70%
Alternative 4	85%	81%	96%	92%
Alternative 5	69%	65%	78%	70%
Alternative 6	57%	50%	65%	54%
Alternative 7	20%	21%	32%	34%
Alternative 8	19%	25%	22%	35%
Alternative 9	23%	22%	35%	35%
Alternative 10	7%	9%	14%	22%

Notes:
[1] Reported relative to Alternative 1.

Abbreviations:
MCL = Maximum Contaminant Level (excludes naphthalene)
MG = millions of gallons of groundwater
PRG = Preliminary Remediation Goal

Table A-7 Evaluation of Remedial Alternatives by COC

Quendall Terminals
Renton, Washington

Alternative	Plume Volume (MG) Benzene	Plume Volume (MG) Naphthalene	Plume Volume (MG) B[a]P	Plume Volume (MG) Arsenic	Plume Contaminant Mass (kg) Benzene	Plume Contaminant Mass (kg) Naphthalene	Plume Contaminant Mass (kg) B[a]P	Plume Contaminant Mass (kg) Arsenic
Pre-Remediation	226	292	23.3	31.6	317	990	5.71	4.54
Alternative 1	226	292	27.0	55.2	317	990	6.10	4.50
Alternative 2	196	262	26.6	54.5	284	907	6.04	4.61
Alternative 3	142	215	23.4	52.4	236	689	5.15	4.47
Alternative 4	181	256	25.3	54.0	191	789	5.46	4.61
Alternative 5	137	207	18.5	50.8	155	471	3.12	4.26
Alternative 6	98.8	171	14.4	48.4	98	258	1.55	3.96
Alternative 7	6.83	33.5	5.99	43.8	0.80	1.29	0.09	3.40
Alternative 8	0.00	0.00	18.0	49.3	0.01	0.00	0.49	3.92
Alternative 9	7.58	40.1	5.10	43.5	3.17	4.26	0.09	3.21
Alternative 10	0.00	0.00	0.00	19.1	0.00	0.00	0.04	2.10

Alternative	Mass Flux at Mudline (kg/year) Benzene	Mass Flux at Mudline (kg/year) Naphthalene	Mass Flux at Mudline (kg/year) B[a]P	Mass Flux at Mudline (kg/year) Arsenic	Restoration Timeframe (years) Benzene	Restoration Timeframe (years) Naphthalene	Restoration Timeframe (years) B[a]P	Restoration Timeframe (years) Arsenic
Pre-Remediation	292	363	2	Not Estimated	>100	>100	>100	>100
Alternative 1	292	363	2.0	5.2	>100	>100	>100	>100
Alternative 2	213	252	1.5	4.9	>100	>100	>100	>100
Alternative 3	127	153	0.9	5.0	>100	>100	>100	>100
Alternative 4	76	140	0.3	5.2	>100	>100	>100	>100
Alternative 5	58	71	0.2	4.9	>100	>100	>100	>100
Alternative 6	40	39	0.1	4.9	>100	>100	>100	>100
Alternative 7	0.4	0.4	0.01	4.9	>100	>100	>100	>100
Alternative 8	0.03	0.01	0.03	4.8	28	98	>100	>100
Alternative 9	0.2	0.1	0.00	2.0	>100	>100	>100	>100
Alternative 10	0.00	0.00	0.00	0.7	14	46	0[2]	>100

Notes:

[1] Reported relative to Alternative 1.

[2] Modeling results do not include the potential contribution of residuals resulting from removal actions (i.e., excavation or dredging). It is expected, based on a model sensitivity analysis (see Appendix A, Section A5.1.2.2), that residuals will result in benzo(a)pyrene exceedances after 100 years for all alternatives, including Alternative 10.

Abbreviations:

B[a]P = benzo(a)pyrene
COC = chemical of concern
kg = kilograms
kg/year = kilograms per year
MG = millions of gallons of groundwater

Table A-7 Evaluation of Remedial Alternatives by COC

Quendall Terminals
Renton, Washington

Alternative	Plume Volume as Percent[1] Benzene	Naphthalene	B[a]P	Arsenic	Plume Contaminant Mass as Percent[1] Benzene	Naphthalene	B[a]P	Arsenic
Alternative 2	86%	90%	99%	99%	90%	92%	99%	103%
Alternative 3	63%	74%	87%	95%	74%	70%	84%	99%
Alternative 4	80%	88%	94%	98%	60%	80%	90%	102%
Alternative 5	60%	71%	69%	92%	49%	48%	51%	95%
Alternative 6	44%	59%	53%	88%	31%	26%	25%	88%
Alternative 7	3%	11%	22%	79%	0%	0%	2%	76%
Alternative 8	0%	0%	67%	89%	0%	0%	8%	87%
Alternative 9	3%	14%	19%	79%	1%	0%	1%	71%
Alternative 10	0%	0%	0%[2]	35%	0%	0%	1%	47%

Alternative	Mass Flux at Mudline as Percent[1] Benzene	Naphthalene	B[a]P	Arsenic
Alternative 2	73%	69%	73%	95%
Alternative 3	43%	42%	44%	97%
Alternative 4	26%	39%	17%	101%
Alternative 5	20%	19%	11%	95%
Alternative 6	14%	11%	6%	95%
Alternative 7	0%	0%	1%	94%
Alternative 8	0%	0%	1%	94%
Alternative 9	0%	0%	0%	38%
Alternative 10	0%	0%	0%	14%

Table A-8 Groundwater Discharge to Lake Washington

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Quendall Terminals
Renton, Washington

	Darcy Flux to Lake Washington (cm/s)		Darcy Flux to Lake Washington (cm/year)	
	Nearshore	Offshore	Nearshore	Offshore
Current Conditions (No remedial technologies implemented)				
Maximum	1.7E-05	3.1E-06	543	96.8
Average	5.6E-06	2.4E-06	177[1]	74.6
Upland Capping				
Maximum	1.4E-05	3.0E-06	427	94.4
Average	4.7E-06	2.3E-06	147[2]	72.8
Upland Capping and Funnel and Gate System				
Maximum	1.3E-05	3.1E-06	397	99.2
Average	4.0E-06	2.5E-06	126	78.1

Notes:

[1] Value used to model current conditions and calibrate the UT model (refer to Appendix B2, Section B2-3.2.1.3).

[2] Value used to model nearshore cap conditions using UT model (refer to Appendix B2, Section B-4.2.2.2).

Abbreviations:

cm/s = centimeters per second
cm/year = centimeters per year
UT = University of Texas

Table A-9 Parameter Sensitivity Analysis[1]

Quendall Terminals
Renton, Washington

Decay Sensitivity Analysis[2]

	Half Life (days)			
	Benzene	**Benzo(a)pyrene**	**Naphthalene**	**Arsenic**
High Half Life	3,600	Not Simulated	1,290	Not Simulated
Low Half Life	112	1,484	40	Not Simulated

K_d Sensitivity Analysis[3]

	Hydrostratigraphic Unit	K_d (L/kg)			
		Benzene	**Benzo(a)Pyrene**	**Naphthalene**	**Arsenic**
High K_d	Shallow Alluvium	0.9	4,280	9	145
	Lake Sediments	0.9	4,280	9	145
	Fill	0.27	1,280	2.75	145
	Deeper Alluvium	0.27	1,280	2.75	145
Low K_d	Shallow Alluvium	0.036	171.2	0.36	5.8
	Lake Sediments	0.036	171.2	0.36	5.8
	Fill	0.0108	51.2	0.11	5.8
	Deeper Alluvium	0.0108	51.2	0.11	5.8

Source Concentration Sensitivity Analysis[4]

	Concentration (µg/L)							
				Benzene				
	Benzo(a)pyrene	**Naphthalene**	**Arsenic**	**Zone 1**	**Zone 2**	**Zone 3**	**Zone 4**	**Zone 5**
High Concentration	200	16,000	58	Not Simulated	1,600	Not Simulated	300	17,000
Low Concentration	70	5,300	19	Not Simulated	530	Not Simulated	100	5,800

Notes:

1 Base parameter values are reported in Tables A-1 and A-2.

2 Half Life end members are lowest estimated anaerobic half life (Howard 1991) and 500% of the base values.

3 K_d end members are 500% and 20% of base values.

4 Concentration end members are 50% and 150% of base parameters.

Abbreviations:

L/kg = liters per kilogram
µg/L = micrograms per liter

Table 10 - Sensitivity Analysis Results by COC

Quendall Terminals
Renton, Washington

Sensitivity Model Run	Plume Volume (MG)				Plume Contaminant Mass (kg)			
	Benzene	Naphthalene	B[a]P	Arsenic	Benzene	Naphthalene	B[a]P	Arsenic
Alternative 1								
Base Parameters	226.47	292.13	27.00	55.24	316.57	989.62	6.10	4.50
High Half Life	448.33	643.39	27.00	55.24	406.20	1,577.42	6.10	4.50
Low Half Life	69.63	76.14	26.98	55.24	204.73	597.35	6.09	4.50
High K_d	226.70	290.23	19.50	41.10	316.73	989.20	5.43	4.62
Low K_d	226.45	292.05	49.41	36.40	316.47	989.66	8.12	1.86
High Concentration	254.03	310.81	28.04	64.66	452.22	1,439.55	9.17	6.84
Low Concentration	173.91	251.95	25.08	35.86	152.30	476.66	3.21	1.92
Alternative 7								
Base Parameters	6.68	33.51	5.99	43.85	0.80	1.29	0.09	3.40
High Half Life	14.08	310.95	6.00	43.85	1.62	19.24	0.09	3.40
Low Half Life	1.14	3.03	5.98	43.85	0.23	0.18	0.09	3.40
High K_d	8.00	186.80	1.00	30.27	1.03	44.01	0.01	3.27
Low K_d	6.68	26.50	24.73	31.20	0.80	0.88	0.56	1.71
High Concentration	9.32	42.44	6.72	49.79	1.20	1.93	0.14	5.10
Low Concentration	3.49	19.39	4.95	25.41	0.34	0.57	0.05	1.31
Alternative 8								
Base Parameters	0.01	0.00	18.02	49.30	0.01	0.00	0.49	3.92
High Half Life	0.01	303.12	17.75	49.30	0.00	16.96	0.49	3.92
Low Half Life	0.00	0.00	17.73	49.30	0.00	0.00	0.48	3.92
High K_d	0.01	207.90	8.09	35.53	0.00	54.17	0.12	3.89
Low K_d	0.01	0.00	40.65	38.50	0.01	0.00	1.63	2.17
High Concentration	0.01	0.37	19.07	57.21	0.01	0.00	0.73	5.94
Low Concentration	0.01	0.00	15.36	29.19	0.01	0.00	0.26	1.51

Abbreviations:
B[a]P = Benzo(a)pyrene
K_d = Sorption coefficient
kg = kilograms
MG = millions of gallons of groundwater

Table 10 - Sensitivity Analysis Results by COC

Quendall Terminals
Renton, Washington

Sensitivity Model Run	Mass Flux at Mudline (kg/year)				Restoration Timeframe (years)			
	Benzene	Naphthalene	B[a]P	Arsenic	Benzene	Naphthalene	B[a]P	Arsenic
Alternative 1								
Base Parameters	292.10	363.27	2.05	5.17	>100	>100	>100	>100
High Half Life	327.02	540.29	2.05	5.17	>100	>100	>100	>100
Low Half Life	230.29	208.39	2.04	5.17	>100	>100	>100	>100
High K_d	292.19	362.97	1.87	4.73	>100	>100	>100	>100
Low K_d	291.97	363.29	3.06	4.77	>100	>100	>100	>100
High Concentration	414.14	528.37	3.08	5.55	>100	>100	>100	>100
Low Concentration	141.19	175.03	1.08	4.78	>100	>100	>100	>100
Alternative 7								
Base Parameters	0.43	0.36	0.01	4.86	>100	>100	>100	>100
High Half Life	0.74	4.59	0.01	4.86	>100	>100	>100	>100
Low Half Life	0.24	0.16	0.01	4.86	>100	>100	>100	>100
High K_d	0.53	8.12	0.02	4.62	>100	>100	>100	>100
Low K_d	0.43	0.29	0.15	4.91	>100	>100	>100	>100
High Concentration	0.61	0.53	0.02	4.96	>100	>100	>100	>100
Low Concentration	0.21	0.17	0.01	4.75	>100	>100	>100	>100
Alternative 8								
Base Parameters	0.03	0.01	0.03	4.85	28	98	>100	>100
High Half Life	0.04	3.07	0.03	4.85	85	>100	>100	>100
Low Half Life	0.02	0.00	0.03	4.85	6	18	>100	>100
High K_d	0.03	7.37	0.00	4.64	85	>100	>100	>100
Low K_d	0.03	0.01	0.23	4.95	17	26	>100	>100
High Concentration	0.03	0.00	0.04	4.95	29	>100	>100	>100
Low Concentration	0.03	0.01	0.02	4.74	24	90	>100	>100

Abbreviations:
B[a]P = Benzo(a)pyrene
K_d = Sorption coefficient
kg = kilograms
MG = millions of gallons of groundwater

Table A-11 Sensitivity Analysis Results - Aggregate Plume Volume

Quendall Terminals
Renton, Washington

Sensitivity Model Run	Aggregate Plume Volume (MG)			
	All Aquifers Combined		Upland Deep Aquifer	
	Exceeds PRGs	Exceeds MCLs[1]	Exceeds PRGs	Exceeds MCLs[1]
Alternative 1				
Base Parameters	323	241	73	47
High Half Life	651	462	79	50
Low Half Life	108	96	35	27
High K_d	323	237	75	48
Low K_d	319	237	70	42
High Concentration	351	271	78	52
Low Concentration	269	183	66	37
Alternative 7				
Base Parameters	65	52	23	16
High Half Life	324	56	40	16
Low Half Life	49	49	16	16
High K_d	193	36	47	14
Low K_d	61	54	16	12
High Concentration	76	59	29	20
Low Concentration	40	31	12	8
Alternative 8				
Base Parameters	61	61	16	16
High Half Life	322	60	22	16
Low Half Life	60	60	16	16
High K_d	216	40	44	15
Low K_d	71	71	13	13
High Concentration	69	69	21	21
Low Concentration	40	40	8	8

Notes:
[1] Naphthalene is excluded because it does not have an MCL.

Abbreviations:
MCL = Maximum Contaminant Level
MG = millions of gallons of groundwater
PRG = Preliminary Remediation Goal

Table A-12 Estimated Sensitivity Analysis Results - Aggregate Plume Volume

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Quendall Terminals
Renton, Washington

Alternative	Best Case Aggregate Plume Volume in MG			
	All Aquifers Combined		Upland Deep Aquifer	
	Exceeds PRGs	Exceeds MCLs	Exceeds PRGs	Exceeds MCLs
Alternative 1	108	96	34.6	27.2
Alternative 2	98	86	33.2	25.1
Alternative 3	84	70	27.1	18.2
Alternative 4	95	81	33.2	24.9
Alternative 5	82	68	27.2	18.3
Alternative 6	72	57	22.9	13.4
Alternative 7	40	31	12.4	7.5
Alternative 8	40	40	7.8	7.8
Alternative 9	43	35	12.8	7.6
Alternative 10	30	24	5.5	3.7

Alternative	Worst Case Aggregate Plume Volume in MG			
	All Aquifers Combined		Upland Deep Aquifer	
	Exceeds PRGs	Exceeds MCLs	Exceeds PRGs	Exceeds MCLs
Alternative 1	651	462	78.8	52.3
Alternative 2	606	397	76.6	48.8
Alternative 3	538	292	68.4	37.6
Alternative 4	588	363	76.7	48.5
Alternative 5	526	278	68.6	37.7
Alternative 6	476	204	62.7	29.6
Alternative 7	324	59	46.7	19.6
Alternative 8	322	70.5	43.7	20.8
Alternative 9	337	58.9	49.1	20.2
Alternative 10	271	0.0	39.2	13.8

Notes:
Values shaded in grey are estimated sensitivity results as described in Section A3.7.
Modeling results do not include the potential contribution of residuals resulting from removal actions (i.e., excavation or dredging). It is expected, based on a model sensitivity analysis (see Appendix A, Section A5.1.2.2), that residuals will result in benzo(a)pyrene exceedances after 100 years for all alternatives, including Alternative 10.

Abbreviations:
MCL = Maximum Contaminant Level
MG = millions of gallons of groundwater
PRG = Preliminary Remediation Goal

Table A-13 Estimated Sensitivity Results by COC - Plume Volume

Quendall Terminals
Renton, Washington

Alternative	Best Case Plume Volume (MG)			
	Benzene	Naphthalene	Benzo[a]pyrene	Arsenic
Alternative 1	70	76	20	36
Alternative 2	60	68	18	35
Alternative 3	43	55	15	33
Alternative 4	55	66	17	34
Alternative 5	42	53	11	31
Alternative 6	30	43	7	29
Alternative 7	1	3	1	25
Alternative 8	0	0	8	29
Alternative 9	2	8	0	25
Alternative 10	0	0	0	2

Alternative	Worst Case Worst Case Plume Volume in MG			
	Benzene	Naphthalene	Benzo[a]pyrene	Arsenic
Alternative 1	448	643	49	65
Alternative 2	388	605	50	64
Alternative 3	281	548	46	61
Alternative 4	358	598	48	63
Alternative 5	271	538	40	59
Alternative 6	196	495	35	56
Alternative 7	14	311	25	50
Alternative 8	0	303	41	57
Alternative 9	15	336	24	49
Alternative 10	0	288	0	18

Notes:

Values shaded in grey are estimated sensitivity results as described in Section A3.7.

Benzo[a]pyrene is expected to restore immediately following implementation of Alternative 10; therefore, benzo[a]pyrene plume volume is assumed to be 0 MG for Alternative 10.

Modeling results do not include the potential contribution of residuals resulting from removal actions (i.e., excavation or dredging). It is expected, based on a model sensitivity analysis (see Appendix A, Section A5.1.2.2), that residuals will result in benzo(a)pyrene exceedances after 100 years for all alternatives, including Alternative 10.

Values are rounded to the nearest whole number.

Abbreviations:

MCL = Maximum Contaminant Level
MG = millions of gallons of groundwater
PRG = Preliminary Remediation Goal

Table A-14 Estimated Sensitivity Results by COC - Plume Mass

Quendall Terminals
Renton, Washington

Alternative	Best Case Plume Mass (kg)			
	Benzene	Naphthalene	Benzo[a]pyrene	Arsenic
Alternative 1	152	477	3	2
Alternative 2	137	437	3	2
Alternative 3	113	332	3	2
Alternative 4	92	380	3	2
Alternative 5	74	227	2	2
Alternative 6	47	124	1	2
Alternative 7	0	0	0	1
Alternative 8	0	0	0	2
Alternative 9	1	2	0	1
Alternative 10	0	0	0	1

Alternative	Worst Case Worst Case Plume Mass (kg)			
	Benzene	Naphthalene	Benzo[a]pyrene	Arsenic
Alternative 1	452	1,577	9	7
Alternative 2	406	1,450	9	7
Alternative 3	337	1,112	8	7
Alternative 4	272	1,268	8	7
Alternative 5	221	776	5	6
Alternative 6	140	447	3	6
Alternative 7	2	44	1	5
Alternative 8	0	54	2	6
Alternative 9	5	55	1	5
Alternative 10	0	48	0	3

Notes:
Values shaded in grey are estimated sensitivity results as described in Section A3.7.
Benzo[a]pyrene is expected to restore immediately following implementation of Alternative 10; therefore, benzo[a]pyrene plume mass is assumed to be 0 kg for Alternative 10.
Values are rounded to the nearest whole number.

Abbreviations:
kg = kilograms
MCL = Maximum Contaminant Level
PRG = Preliminary Remediation Goal

Table A-15 Estimated Sensitivity Results by COC - Mass Flux

Quendall Terminals
Renton, Washington

Alternative	Best Case Mass Flux (kg/year)			
	Benzene	Naphthalene	Benzo[a]pyrene	Arsenic
Alternative 1	141	175	1	5
Alternative 2	103	121	1	5
Alternative 3	61	74	0	5
Alternative 4	37	68	0	5
Alternative 5	28	34	0	5
Alternative 6	19	19	0	5
Alternative 7	0	0	0	5
Alternative 8	0	0	0	5
Alternative 9	0	0	0	4
Alternative 10	0	0	0	3

Alternative	Worst Case Mass Flux (kg/year)			
	Benzene	Naphthalene	Benzo[a]pyrene	Arsenic
Alternative 1	414	540	3	6
Alternative 2	302	377	2	5
Alternative 3	180	232	1	5
Alternative 4	108	213	1	6
Alternative 5	82	111	0	5
Alternative 6	56	65	0	5
Alternative 7	1	8	0	5
Alternative 8	0	7	0	5
Alternative 9	0	8	0	4
Alternative 10	0	7	0	3

Notes:
Values shaded in grey are estimated sensitivity results as described in Section A3.7.
Values are rounded to the nearest whole number.

Abbreviations:
kg/year = kilograms per year
MCL = Maximum Contaminant Level
PRG = Preliminary Remediation Goal

Table A-16 Dewatering Estimates - Wet Excavation

Quendall Terminals
Renton, Washington

Excavation Cell	Area in Square Feet	Maximum Excavation Depth (feet bgs)	Estimated Dewatering Flow Rate (gpm)	Dewatering Depth (feet bgs)		Sheet Pile Embedment Depth (feet bgs)		Number of Wells
				Minimum	Maximum	Minimum	Maximum	
1	13,343	34	91	19	21	47	51	6
2	7,985	22	0	-	-	-	-	-
3	13,7060	14	0	-	-	-	-	-
4	80,281	18	137	9	20	38	49	4
5	12,790	24	0	-	-	-	-	-
6	5,541	27	68	13	19	53	62	3
7*	84,507	22	207	9	19	47	65	6
8	11,746	19	47	8	13	50	89	3
9	20,084	15	0	-	-	-	-	-
10	30,708	32	119	19	22	36	37	5
DNAPL Trench**	2,500	25	16	13	14	46	50	2
Quendall Pond	21,556	19	119	8	15	57	64	6

Notes:

* Excavation Cells 1 through 10 and the Quendall Pond excavation are included in Alternative 8.

** The DNAPL Trench is the collection trench included in Alternatives 3 through 7.

- The dash indicates that depressurization of the Deep Aquifer was not required.

Abbreviations:

bgs = below ground surface
gpm = gallons per minute

Table A-17 Dewatering Estimates - Dry Excavation

Quendall Terminals
Renton, Washington

Excavation Cell	Area in square feet	Maximum Excavation Depth (feet bgs)	Estimated Dewatering Flow Rate (gpm)	Dewatering Depth (feet bgs)		Sheet Pile Embedment Depth (feet bgs)		Number of Wells
				Minimum	Maximum	Minimum	Maximum	
1	13,343	34	202	34	38	47	51	6
2	7,985	22	94	22	25	44	49	2
3	13,7060	14	301	14	20	40	59	8
4	80,281	18	462	19	27	38	49	7
5	12,790	24	171	24	29	51	59	4
6	5,541	27	143	27	31	53	62	3
7	84,507	22	592	22	32	44	49	6
8*	11,746	19	143	19	23	50	89	3
9	20,084	15	119	15	19	50	53	3
10	30,708	32	228	32	34	45	47	6
DNAPL Trench**	2,500	26	50	25	27	46	50	2

Notes:
* Excavation Cells 1 through 10 are included in Alternative 8.
** DNAPL Trench is the collection trench included in Alternatives 3 through 7.

Abbreviations:
bgs = below ground surface
gpm = gallons per minute

Table A-18 Fine Grain Layers in the Deeper Alluvium

Quendall Terminals
Renton, Washington

	Silty Sand Lens		Silt Lens	
Boring ID	Depth to Top	Depth to Bottom	Depth to Top	Depth to Bottom
BH-5B	--	--	49.5	50
BH-19B	--	--	45.3	45.5
BH-20C	53	55.5	--	--
	62	62.5	--	--
	73.5	74.5	--	--
BH-21B	43	50	38	39.5
BH-29B	45	46	--	--
BH-30C	--	--	45.8	46.2
SWB-3	33	42	--	--
SWB-4B	33.5	39	--	--
SWB-8	51	52	--	--
	61	83	--	--

Notes:
Depths are reported in feet below ground surface.
Dashes indicate layer not found in present log

Table A19 - Restoration Potential Fate and Transport Model Results

Quendall Terminals
Renton, Washington

Sensitivity Analysis Scenario	Deeper Alluvium Assumption[1]	Pump and Treat[2]	Model Results - Time to Reach MCLs or PRGs[3] in Years[4] or Maximum Concentration			
			Naphthalene	Benzene	Benzo(a)pyrene	Arsenic
Comparison of Heterogeneous and Homogeneous Assumptions	Heterogeneous	None	45	30	> 100 Years (20 µg/L)	> 100 Years (53 µg/L)
		90 gpm	45	26	> 100 Years (20 µg/L*)	> 100 Years (53 µg/L)
		180 gpm	--	25	> 100 Years (20 µg/L*)	> 100 Years (53 µg/L)
	Homogeneous	None	53	13	> 250 Years (4.2 µg/L)	> 200 Years (33 µg/L)
		90 gpm	51	14	> 200 Years (3.8 µg/L)	> 200 Years (30 µg/L)
		180 gpm	--	14	> 200 Years (3.5 µg/L)	> 200 Years (16 µg/L)
Excavation Residual Analysis	Homogeneous	None	--	13	> 100 Years (1.3 µg/L)	--
		90 gpm	--	13	> 100 Years (3.3 µg/L)[5]	--

Notes:

-- Model scenario was not performed for indicated COC.

* Simulation used 30 µg/L as initial condition in the low-permeability layers and negligible reduction observed.
Reported result assumes initial concentration of 20 µg/L would also exhibit negligible reduction.

[1] Model runs that simulate a heterogeneous Deeper Alluvium include a representative silt and silty sand zone.

[2] Total pump and treat flow rate from 6 pumping wells near the shoreline.

[3] Naphthalene PRG = 1.4 µg/L, benzene MCL = 5 µg/L, benzo(a)pyrene MCL = 0.2 µg/L, and arsenic MCL = 10 µg/L.

[4] The maximum concentration at the end of the simulation is reported when the COC does not attenuate below the MCL within the modeled timeframe.

[5] A greater remaining concentration was observed with pumping because of stagnation created by pumping.

Abbreviations:

gpm = gallons per minute
MCL = Maximum Contaminant Level
MTCA = Model Toxics Control Act
µg/L = micrograms per liter

Table A-20 Dewatering Estimates for Locations near the Railroad Area and Shoreline

Quendall Terminals
Renton, Washington

Near Rail Road Area

Excavation Area	Sheet Pile Embedment Depth (bgs)	Dewater Depth (feet bgs)	Combined Pumping Rate (gpm)
2 Acres	1.5 x Shallow Alluvium Thickness	35 to 40	740
	1.5 x Shallow Alluvium Thickness + 20 Feet	35 to 40	510
	1.5 x Shallow Alluvium Thickness + 40 Feet	35 to 40	360
1 Acre	1.5 x Shallow Alluvium Thickness	35 to 40	570
	1.5 x Shallow Alluvium Thickness + 20 Feet	35 to 40	310
	1.5 x Shallow Alluvium Thickness + 40 Feet	35 to 40	200
0.5 Acres	1.5 x Shallow Alluvium Thickness	35 to 40	330
	1.5 x Shallow Alluvium Thickness + 20 Feet	35 to 40	160
	1.5 x Shallow Alluvium Thickness + 40 Feet	35 to 40	110
0.25 Acres	1.5 x Shallow Alluvium Thickness	35 to 40	180
	1.5 x Shallow Alluvium Thickness + 20 Feet	35 to 40	79
	1.5 x Shallow Alluvium Thickness + 40 Feet	35 to 40	52

Abbreviations:
bgs = below ground surface
gpm = gallons per minute

Table A-20 Dewatering Estimates for Locations near the Railroad Area and Shoreline

Quendall Terminals
Renton, Washington

Near the Shoreline

Excavation Area	Sheet Pile Embedment Depth bgs	Dewater Depth (feet bgs)	Combined Pumping Rate (gpm)
2 Acres	1.5 x Shallow Alluvium Thickness	35 to 45	940
	1.5 x Shallow Alluvium Thickness + 20 Feet	20 to 25 35 to 45	320 570
	1.5 x Shallow Alluvium Thickness + 40 Feet	20 to 25 35 to 45	210 400
1 Acre	1.5 x Shallow Alluvium Thickness	20 to 30 35 to 45	380 680
	1.5 x Shallow Alluvium Thickness + 20 Feet	20 to 30 35 to 45	210 350
	1.5 x Shallow Alluvium Thickness + 40 Feet	20 to 30 35 to 45	130 210
0.5 Acres	1.5 x Shallow Alluvium Thickness	20 to 30 35 to 45	230 400
	1.5 x Shallow Alluvium Thickness + 20 Feet	20 to 30 35 to 45	110 190
0.25 Acres	1.5 x Shallow Alluvium Thickness	20 to 30 35 to 45	110 210
	1.5 x Shallow Alluvium Thickness + 20 Feet	20 to 30 35 to 45	52 94

Abbreviations:
bgs = below ground surface
gpm = gallons per minute

Legend

DNAPL Thiessen Polygon

Hydrocarbon Source Zone

Detention Pond

Existing Structure

Historical Structure

Other Historical Feature

Current Shoreline

Maximum DNAPL Depth in Feet

Imported to model as depth of DNAPL constant concentration boundary condition (i.e., hydrocarbon source).

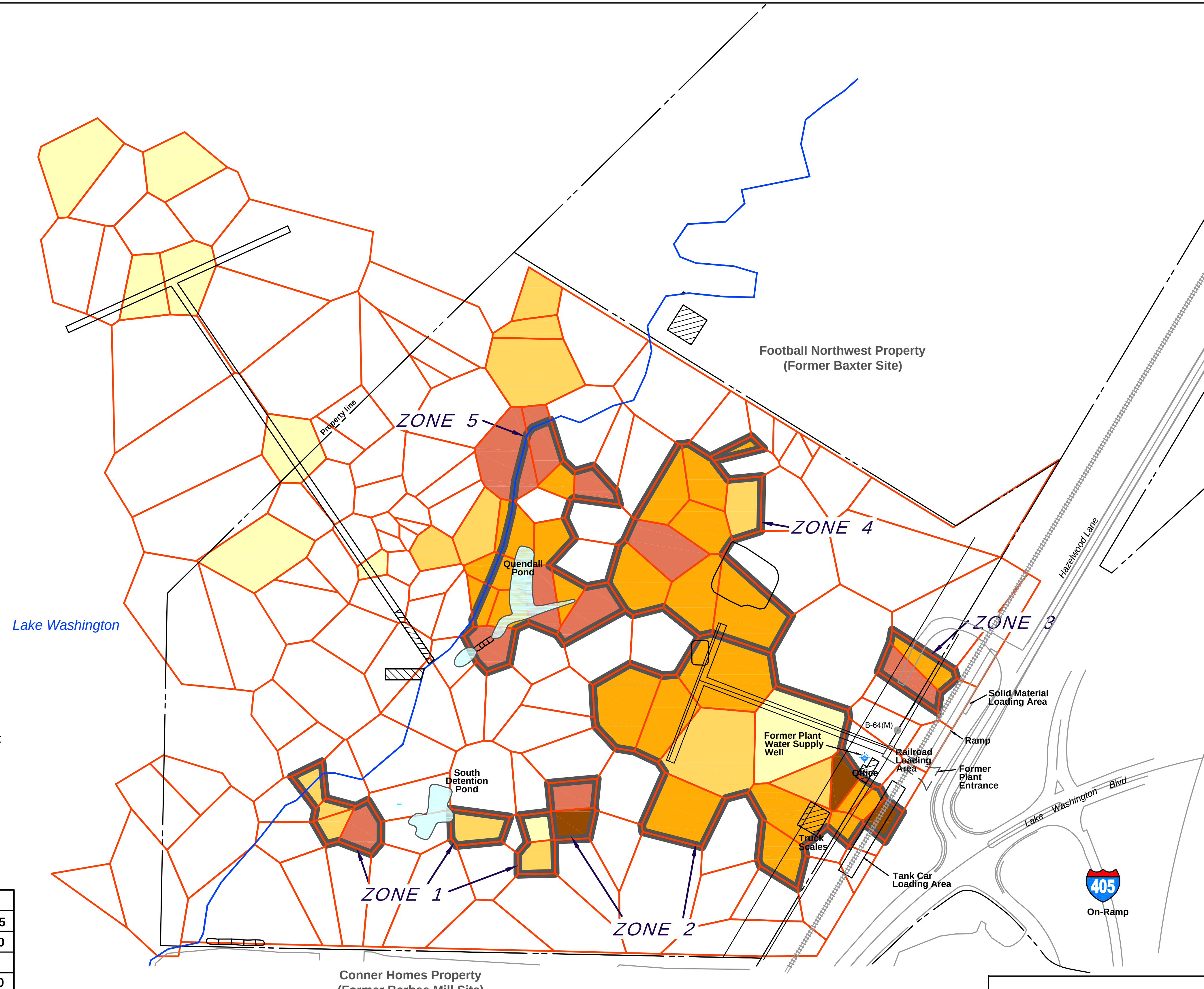
6.0

12.0

18.0

24.0

33.0



Notes

1. Modified from Figure 4.4-5 of Quendall Terminals RI Report (Anchor QEA and Aspect 2012).
2. Thiessen polygons based on midpoint between borings of adequate depth and characterization, truncated at property line.
3. See Appendix G, Figure G-1, and Table G-5 of the RI Report for maximum depth and area for each polygon.
4. DNAPL identified as oil-coated or oil-wetted soil. Sheen and stained soil not identified as DNAPL. See Section 4.3.1 for DNAPL definitions and Tables G-1 through G-4 in Appendix G of the RI Report for summaries of DNAPL characterization at each boring.

Source Concentration by Zone in µg/L

COC	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5
Benzene	NS	1,100	NS	200	12,000
Benzo(a)pyrene	133	133	NS	133	133
Naphthalene	11,000	11,000	NS	11,000	11,000

NS = Not Simulated

N

0 200 400

Feet

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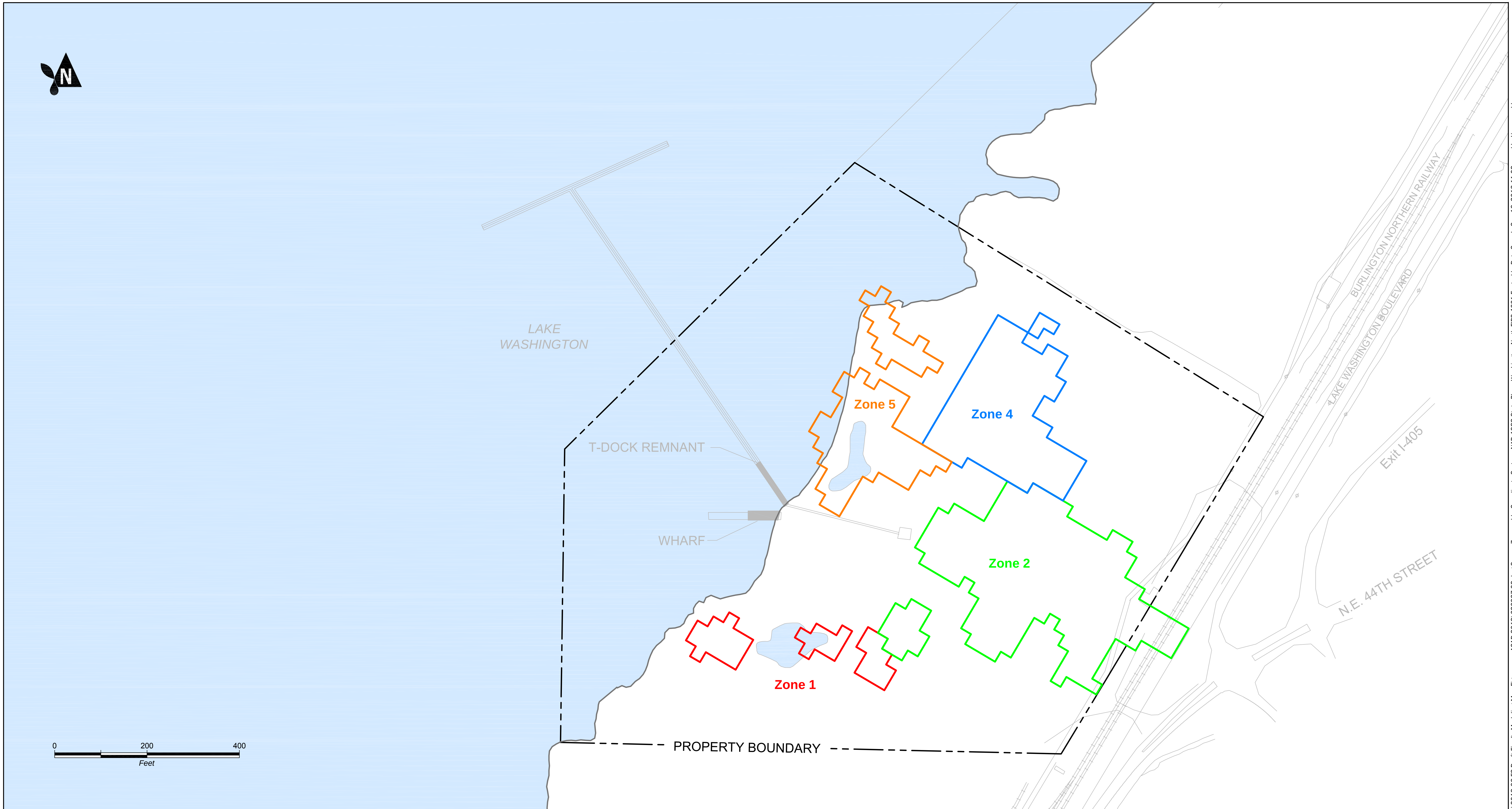
Hydrocarbon Source Zones

Quendall Terminals Feasibility Study Report

Renton, Washington

Aspect CONSULTING | ARCADIS | BY: SDM/PMB REV BY: SCC | FIGURE NO. A-1



Legend

Hydrocarbon Source Zones

Source Concentration by Zone in µg/L					
COC	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5
Benzene	NS	1,100	NS	200	12,000
Benzo(a)pyrene	133	133	NS	133	133
Naphthalene	11,000	11,000	NS	11,000	11,000

NS = Not Simulated

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Modeled Hydrocarbon Source Zones
Plan View

Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING

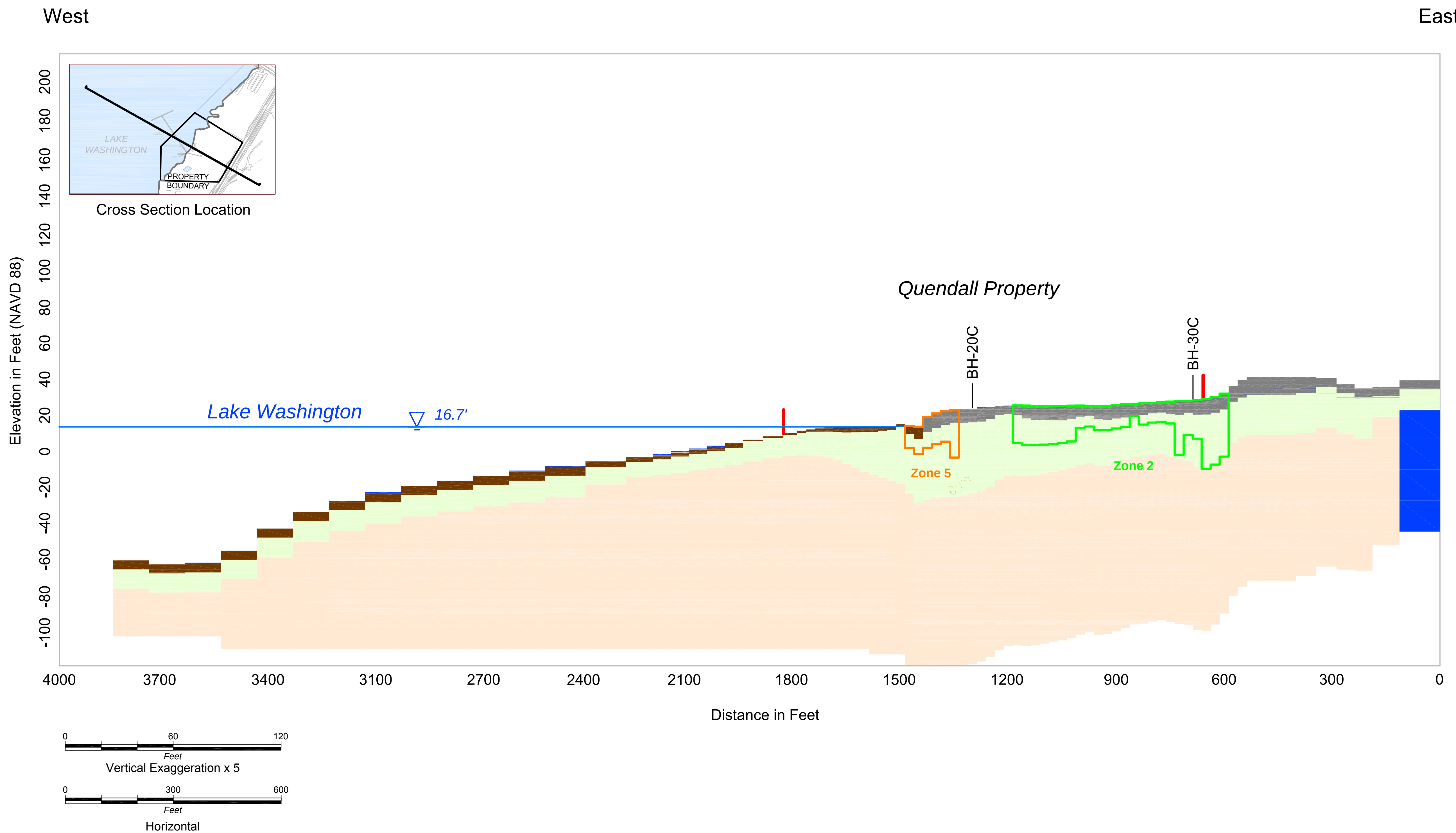
ARCADIS

FIRM: ASPECT

DRAWN BY: SM/SCC

FIGURE NO. A-2

CAD Path: Q:\Quendall\020027 Quendall Terminals\2013-06 FS Draft Final_Modeling Map Figures use ACAD 2012\020027-02.dwg Source Zones || Coordinate System: NAD 1983 State Plane Washington North FIPS 4601 Feet || Date Saved: Sep 16, 2013 11:02am || User: scudd



All elevations are in feet NAVD 88.

Legend

- Fill
- Shallow Alluvium
- Deeper Alluvium
- Lake Washington Sediments
- Constant Head Boundary Cell
- Quendall Property Boundary
- Hydrocarbon Source Zones

Source Concentration by Zone in µg/L					
COC	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5
Benzene	NS	1,100	NS	200	12,000
Benzo(a)pyrene	133	133	NS	133	133
Naphthalene	11,000	11,000	NS	11,000	11,000

NS = Not Simulated
Zones 1 & 4 are not visible because they are not bisected by this cross section.

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Modeled Hydrocarbon Source Zones
Cross Section View
Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS | FIRM: ASPECT | DRAWN BY: SM/SCC | FIGURE NO. A-3

Pre-remediation Plume Extent in Layer 2

- Benzene Plume from Model *(Equal to 5 µg/L Isoconcentration Contour)*[1]
- Benzene Plume from Site Data *(Equal to 5 µg/L Isoconcentration Contour)*[2]

Dashed line indicates estimate based on limited chemical data and groundwater flow paths, and does not include dispersion. See figures 3-6 and 3-8.

Notes:

1. Extents estimated by MODFLOW/MT3D assuming a hydrocarbon source for 100 years.
2. Extents estimated from groundwater data adapted from figure 3-6.
3. Groundwater modeling results should be used as a relative tool for comparison and not considered a prediction of actual conditions. For example, model-estimated benzo[a]pyrene plume volumes for all alternatives are larger than anticipated based on current site data, due to simplifying modeling assumptions (see Appendix A, Section A3.2).

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Model Simulated Pre-remediation Benzene Plume-Plan View

Quendall Terminals Feasibility Study Report
Renton, Washington

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FIRM: ASPECT
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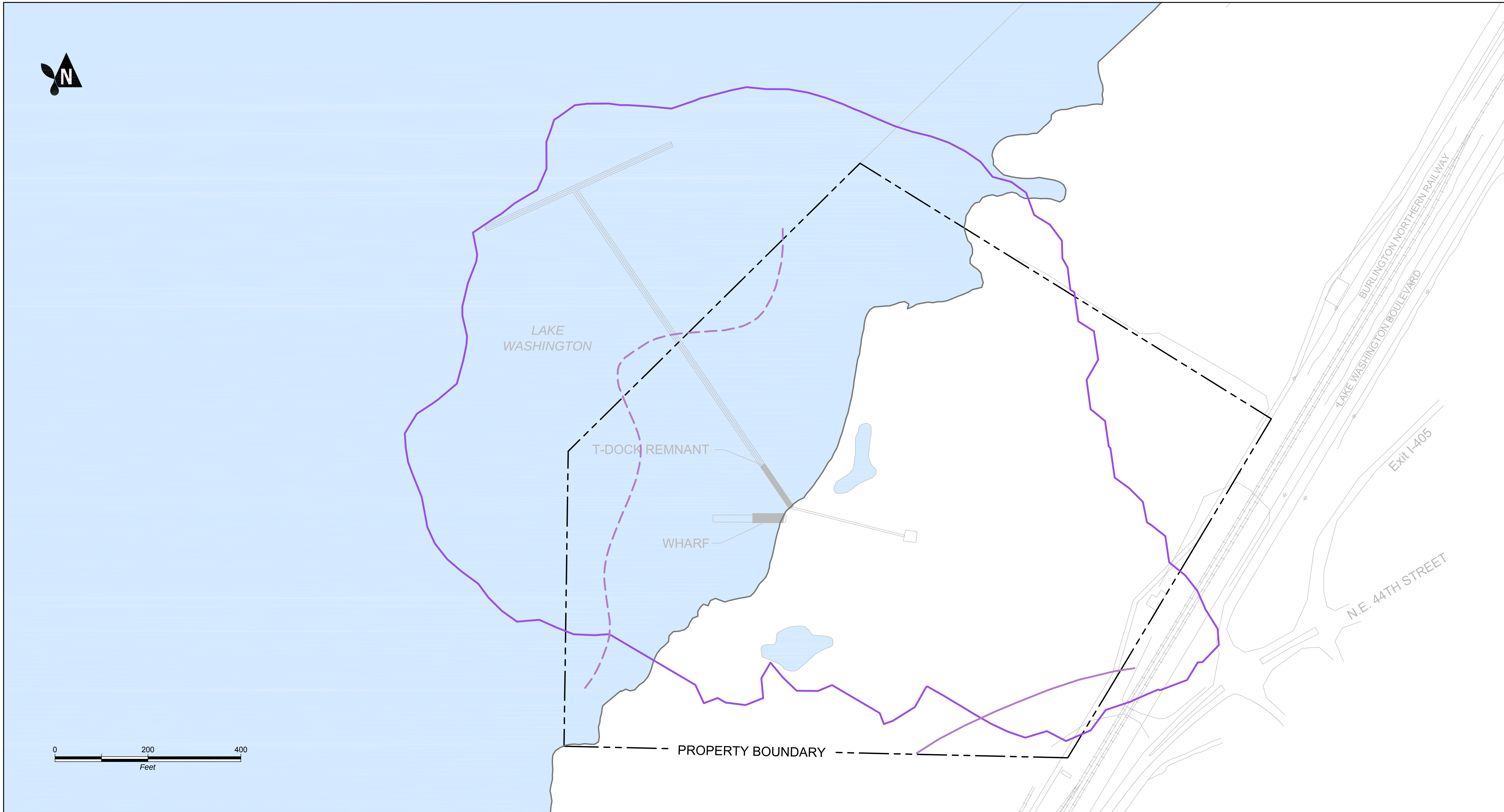
FIGURE NO. **A-4**



Pre-remediation Plume Extent in Layer 2

- Naphthalene Plume from Model *(Equal to 1.4 µg/L Isoconcentration Contour)*[1]
- Naphthalene Plume from Site Data *(Equal to 1.4 µg/L Isoconcentration Contour)*[2]

Dashed line indicates estimate based on limited chemical data and groundwater flow paths, and does not include dispersion. See figures 3-6 and 3-8.

Notes:

1. Extents estimated by MODFLOW/MT3D assuming a hydrocarbon source for 100 years.
2. Extents estimated from groundwater data adapted from figure 3-6.
3. Groundwater modeling results should be used as a relative tool for comparison and not considered a prediction of actual conditions. For example, model-estimated benzo[a]pyrene plume volumes for all alternatives are larger than anticipated based on current site data, due to simplifying modeling assumptions (see Appendix A, Section A3.2).

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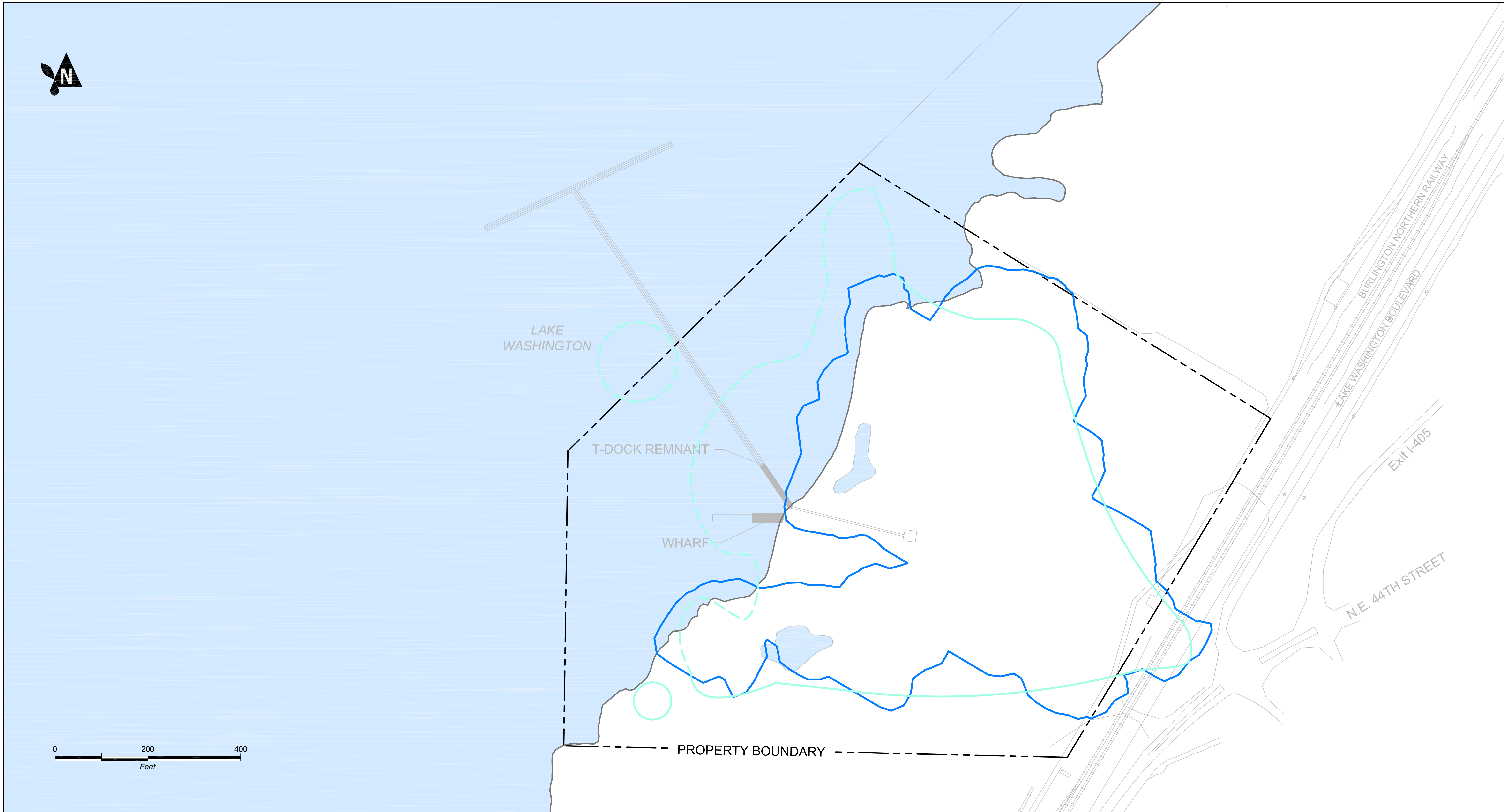
Model Simulated Pre-remediation Naphthalene Plume-Plan View

Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS | FIRM: ASPECT | DRAWN BY: SM/SCC | FIGURE NO. A-5

CAD Path: Q:\Quendall\020027 Quendall Terminals\2013-06 FS Draft Final_Modeling Map Figures use ACAD 2012\020027-08.dwg Naphthalene-Pre-remediation || Coordinate System: NAD 1983 State Plane Washington North FIPS 4601 Feet || Date Saved: Sep 16, 2013 11:11am || User: scudd



Pre-remediation Plume Extent in Layer 2

Benzo(a)pyrene Plume from Model *(Equal to 0.2 µg/L Isoconcentration Contour)*[1]
Benzo(a)pyrene Plume from Site Data *(Equal to 0.2 µg/L Isoconcentration Contour)*[2]

Dashed line indicates estimate based on limited chemical data and groundwater flow paths, and does not include dispersion. See figures 3-6 and 3-8.

Notes:

1. Extents estimated by MODFLOW/MT3D assuming a hydrocarbon source for 100 years.
2. Extents estimated from groundwater data adapted from figure 3-6.
3. Groundwater modeling results should be used as a relative tool for comparison and not considered a prediction of actual conditions. For example, model-estimated benzo[a]pyrene plume volumes for all alternatives are larger than anticipated based on current site data, due to simplifying modeling assumptions (see Appendix A, Section A3.2).

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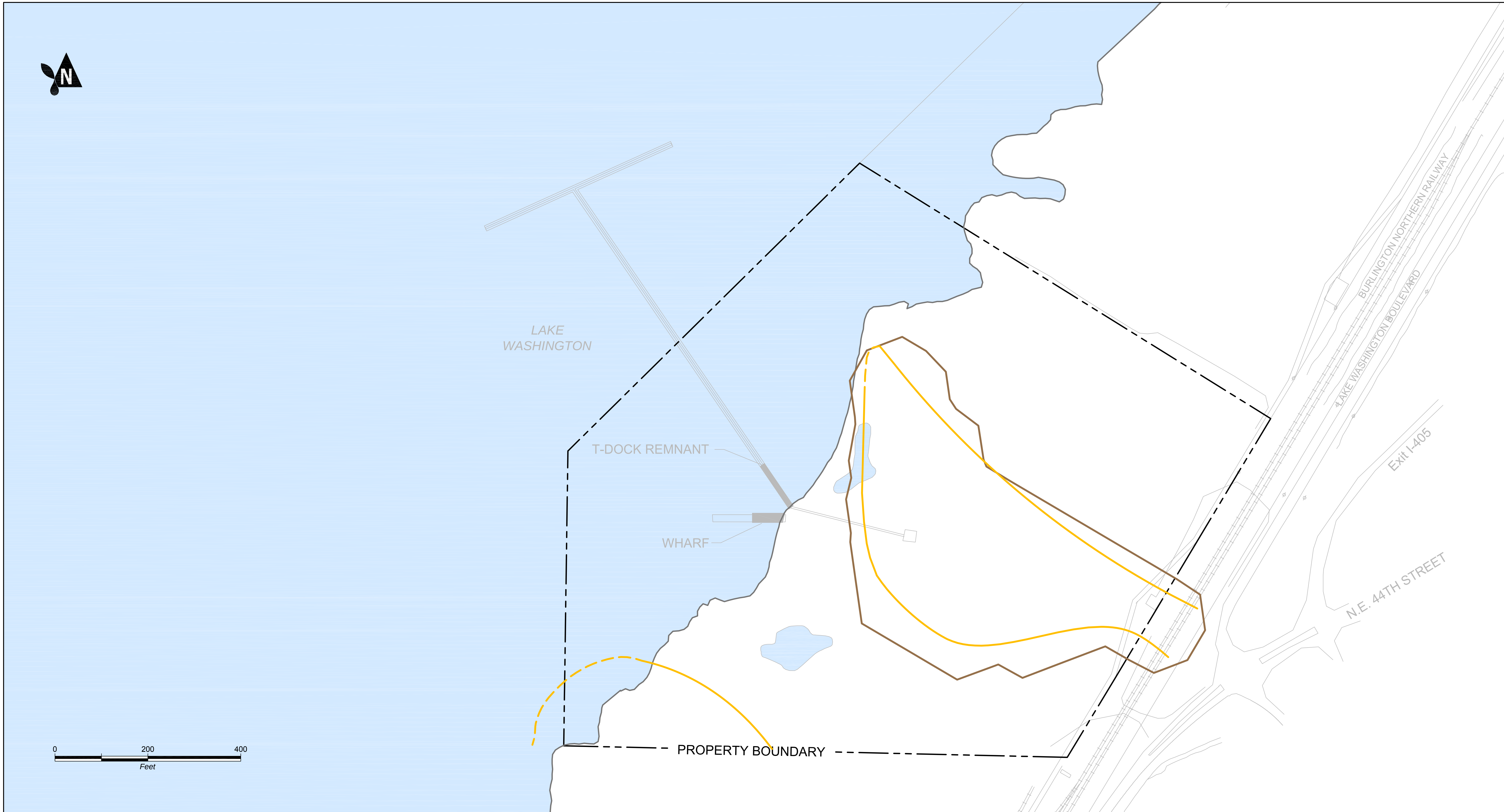
Model Simulated Pre-remediation Benzo(a)pyrene Plume-Plan View
Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS | FIRM: ASPECT | FIGURE NO. A-6
DRAWN BY: SM/SCC

CAD Path: Q:\Quendall\020027 Quendall Terminals\2013-06 FS Draft Final_Modeling Map Figures use ACAD 2012\020027-08.dwg Benzo(a)pyrene-Pre-remediation || Coordinate System: NAD 1983 State Plane Washington North FIPS 4601 Feet || Date Saved: Sep 16, 2013 11:15am || User: scudd



Pre-remediation Plume Extent in Layer 2

Arsenic Plume from Model *(Equal to 10 µg/L Isoconcentration Contour)*[1]

Arsenic Plume from Site Data *(Equal to 10 µg/L Isoconcentration Contour)*[2]

Dashed line indicates estimate based on limited chemical data and groundwater flow paths, and does not include dispersion. See figures 3-6 and 3-8.

Notes:

1. Extents estimated by MODFLOW/MT3D assuming a hydrocarbon source for 100 years.
2. Extents estimated from groundwater data adapted from figure 3-6.
3. Groundwater modeling results should be used as a relative tool for comparison and not considered a prediction of actual conditions. For example, model-estimated benzo[a]pyrene plume volumes for all alternatives are larger than anticipated based on current site data, due to simplifying modeling assumptions (see Appendix A, Section A3.2).

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Model Simulated Pre-remediation Arsenic Plume-Plan View

Quendall Terminals Feasibility Study Report

Renton, Washington

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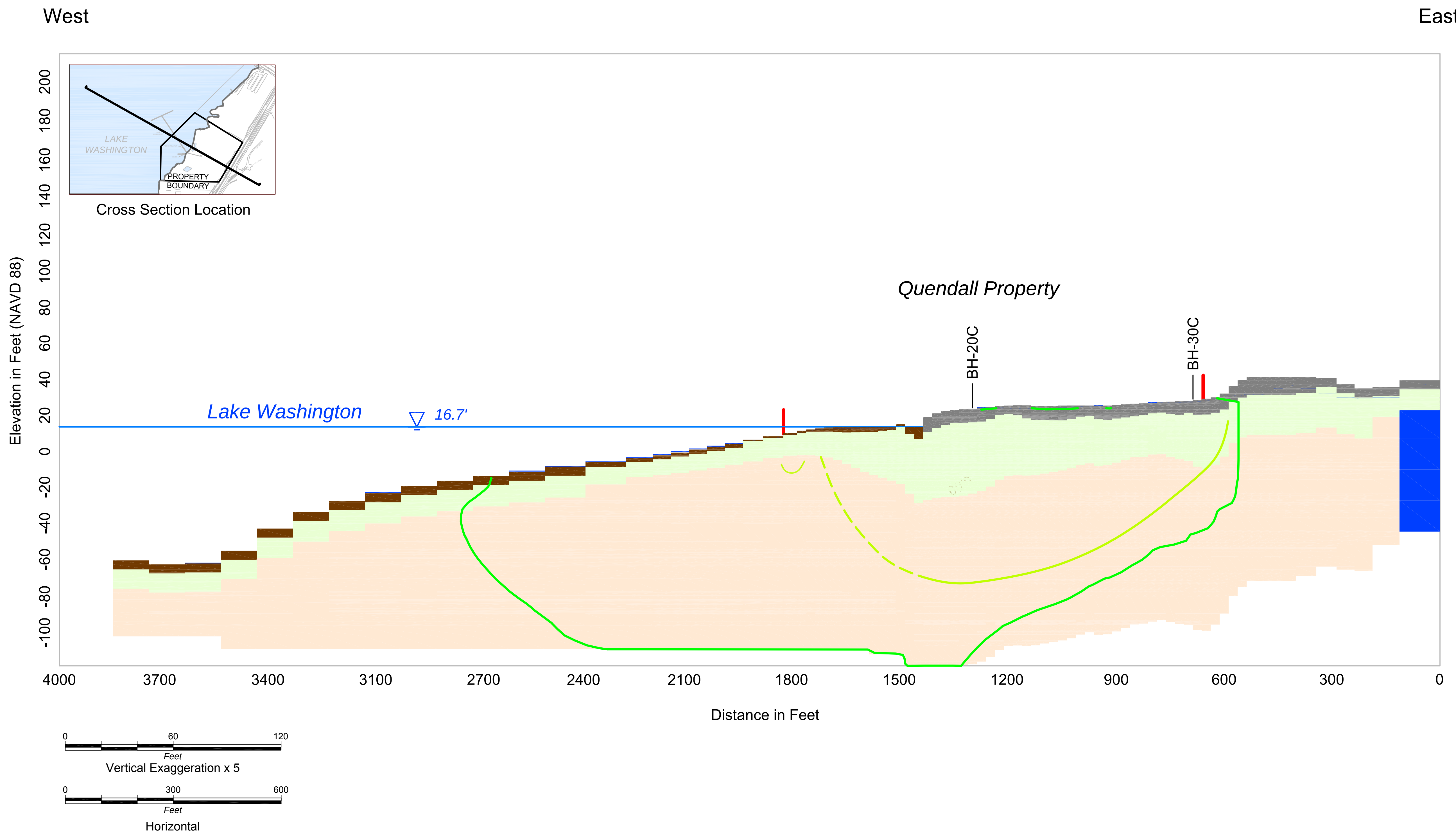
FIGURE NO. **A-7**

CAD Path: Q:\Quendall\020027 Quendall Terminals\2013-06 FS Draft Final_Modeling Map Figures use ACAD 2012\020027-08.dwg Arsenic-Pre-remediation || Coordinate System: NAD 1983 State Plane Washington North FIPS 4601 Feet || Date Saved: Sep 16, 2013 11:16am || User: scudd



All elevations are in feet NAVD 88.

Legend

- Fill
- Shallow Alluvium
- Deeper Alluvium
- Lake Washington Sediments
- Constant Head Boundary Cell
- Quendall Property Boundary

Pre-remediation Plume Extent in Column 76

- Benzene Plume from Model *(Equal to 5 µg/L Isoconcentration Contour)*[1]
- Benzene Plume from Site Data *(Equal to 5 µg/L Isoconcentration Contour)*[2]

Dashed line indicates estimate based on limited chemical data and groundwater flow paths, and does not include dispersion. See figures 3-6 and 3-8.

Notes:

1. Extents estimated by MODFLOW/MT3D assuming a hydrocarbon source for 100 years.
2. Extents estimated from groundwater data adapted from figure 3-8.
3. Groundwater modeling results should be used as a relative tool for comparison and not considered a prediction of actual conditions. For example, model-estimated benzo[a]pyrene plume volumes for all alternatives are larger than anticipated based on current site data, due to simplifying modeling assumptions (see Appendix A, Section A3.2).

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Model Simulated Pre-remediation Benzene Plume-Cross Section View

Quendall Terminals Feasibility Study Report

Renton, Washington

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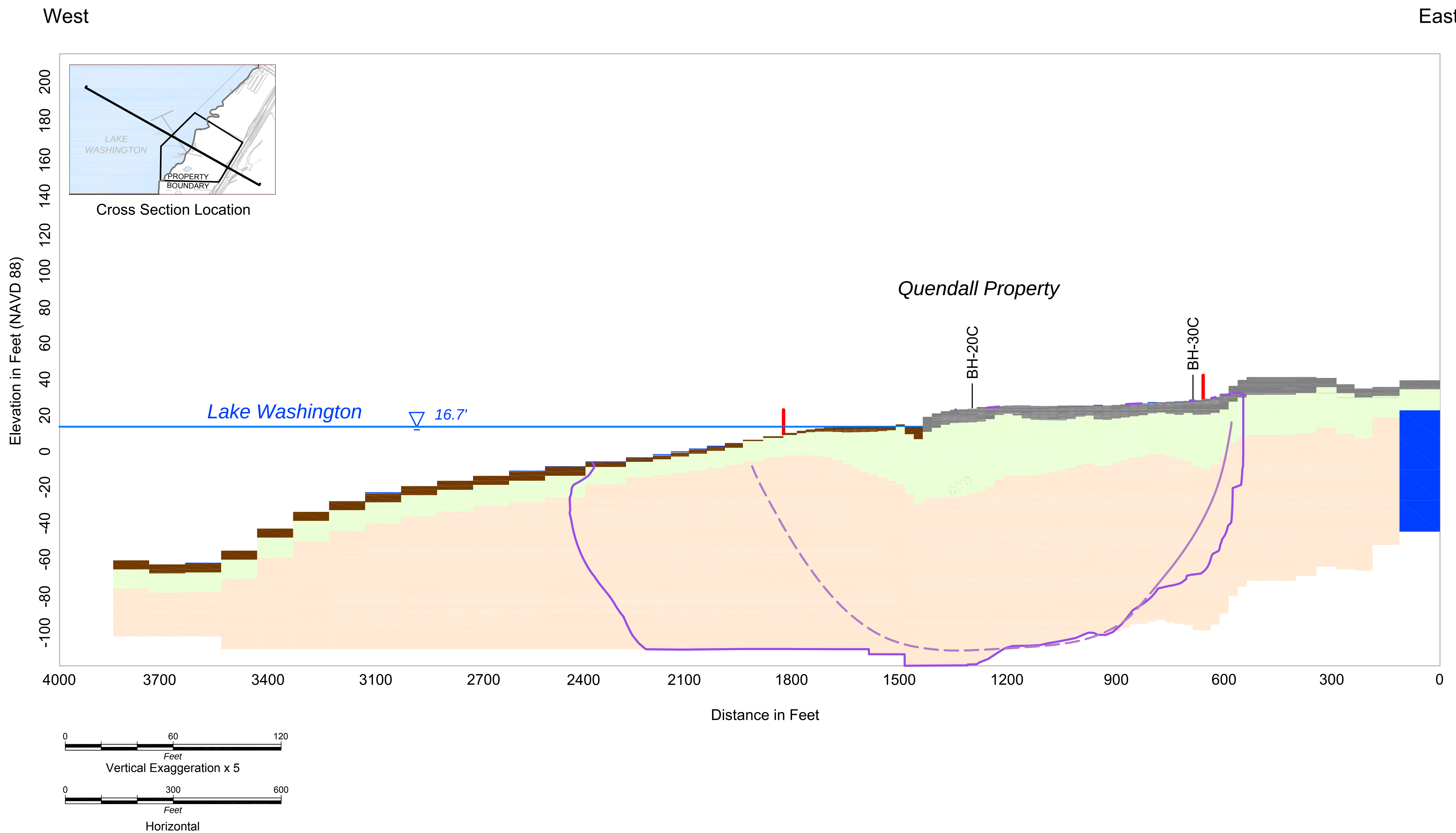
FIRM: ASPECT

DRAWN BY: SM/SCC

FIGURE NO. A-8



All elevations are in feet NAVD 88.

Legend

- Fill
- Shallow Alluvium
- Deeper Alluvium
- Lake Washington Sediments
- Constant Head Boundary Cell
- Quendall Property Boundary

Pre-remediation Plume Extent in Column 76

- Naphthalene Plume from Model *(Equal to 1.4 µg/L Isoconcentration Contour)*[1]
- Naphthalene Plume from Site Data *(Equal to 1.4 µg/L Isoconcentration Contour)*[2]

Dashed line indicates estimate based on limited chemical data and groundwater flow paths, and does not include dispersion. See figures 3-6 and 3-8.

Notes:

1. Extents estimated by MODFLOW/MT3D assuming a hydrocarbon source for 100 years.
2. Extents estimated from groundwater data adapted from figure 3-8.
3. Groundwater modeling results should be used as a relative tool for comparison and not considered a prediction of actual conditions. For example, model-estimated benzo[a]pyrene plume volumes for all alternatives are larger than anticipated based on current site data, due to simplifying modeling assumptions (see Appendix A, Section A3.2).

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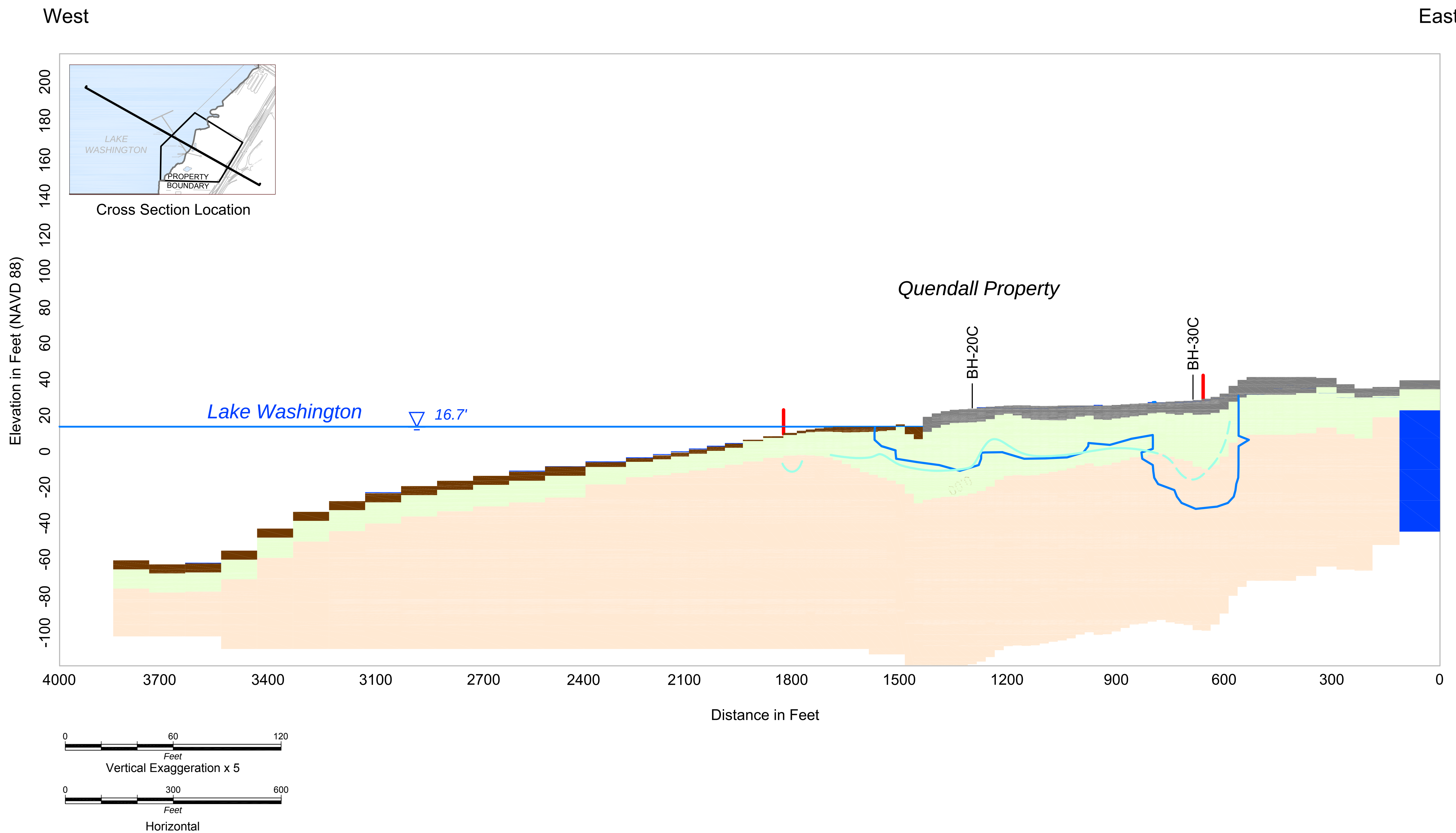
Model Simulated Pre-remediation Naphthalene Plume-Cross Section View

Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS | FIRM: ASPECT | DRAWN BY: SM/SCC | FIGURE NO. A-9



All elevations are in feet NAVD 88.

Legend

- Fill
- Shallow Alluvium
- Deeper Alluvium
- Lake Washington Sediments
- Constant Head Boundary Cell
- Quendall Property Boundary

Pre-remediation Plume Extent in Column 76

- Benzo(a)pyrene Plume from Model *(Equal to 0.2 µg/L Isoconcentration Contour)*[1]
- Benzo(a)pyrene Plume from Site Data *(Equal to 0.2 µg/L Isoconcentration Contour)*[2]

Dashed line indicates estimate based on limited chemical data and groundwater flow paths, and does not include dispersion. See figures 3-6 and 3-8. Dashed extent is based on Site data adjusted based on soil data in the Shallow Alluvium.

Notes:

1. Extents estimated by MODFLOW/MT3D assuming a hydrocarbon source for 100 years. Model may over predict the extent of benzo[a]pyrene in the Deep Aquifer due to modeling artifacts (see section A3.2.3).
2. Extents estimated from groundwater data adapted from figure 3-8.
3. Groundwater modeling results should be used as a relative tool for comparison and not considered a prediction of actual conditions. For example, model-estimated benzo[a]pyrene plume volumes for all alternatives are larger than anticipated based on current site data, due to simplifying modeling assumptions (see Appendix A, Section A3.2).

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Model Simulated Pre-remediation Benzo(a)pyrene Plume-Cross Section View

Quendall Terminals Feasibility Study Report

Renton, Washington

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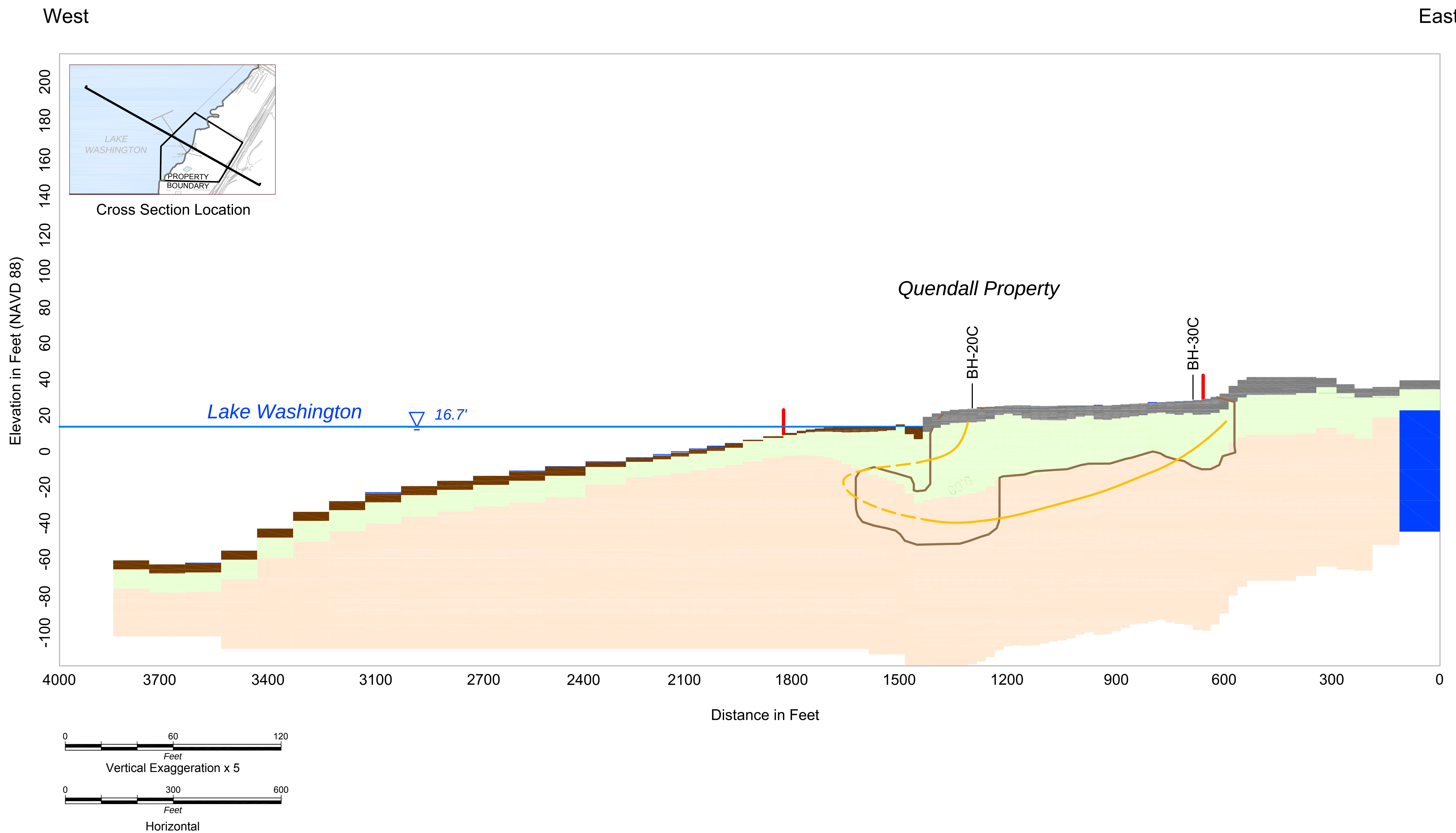
DRAWN BY: SM/SCC

FIGURE NO. A-10



All elevations are in feet NAVD 88.

Legend

- Fill
- Shallow Alluvium
- Deeper Alluvium
- Lake Washington Sediments
- Constant Head Boundary Cell
- Quendall Property Boundary

Pre-remediation Plume Extent in Column 76

- Arsenic Plume from Model *(Equal to 10 µg/L Isoconcentration Contour)*[1]
- Arsenic Plume from Site Data *(Equal to 10 µg/L Isoconcentration Contour)*[2]

Dashed line indicates estimate based on limited chemical data and groundwater flow paths, and does not include dispersion. See figures 3-6 and 3-8.

Notes:

1. Extents estimated by MODFLOW/MT3D assuming a hydrocarbon source for 100 years.
2. Extents estimated from groundwater data adapted from figure 3-8.
3. Groundwater modeling results should be used as a relative tool for comparison and not considered a prediction of actual conditions. For example, model-estimated benzo[a]pyrene plume volumes for all alternatives are larger than anticipated based on current site data, due to simplifying modeling assumptions (see Appendix A, Section A3.2).

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Model Simulated Pre-remediation Arsenic Plume-Cross Section View
Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS | FIRM: ASPECT | DRAWN BY: SM/SCC | FIGURE NO. **A-11**

CAD Path: Q:\Quendall\020027 Quendall Terminals\2013-06 FS Draft Final_Modeling Map Figures use ACAD 2012\020027-10.dwg Arsenic || Coordinate System: NAD 1983 State Plane Washington North FIPS 4601 Feet || Date Saved: Sep 16, 2013 11:38am || User: scudd

Legend

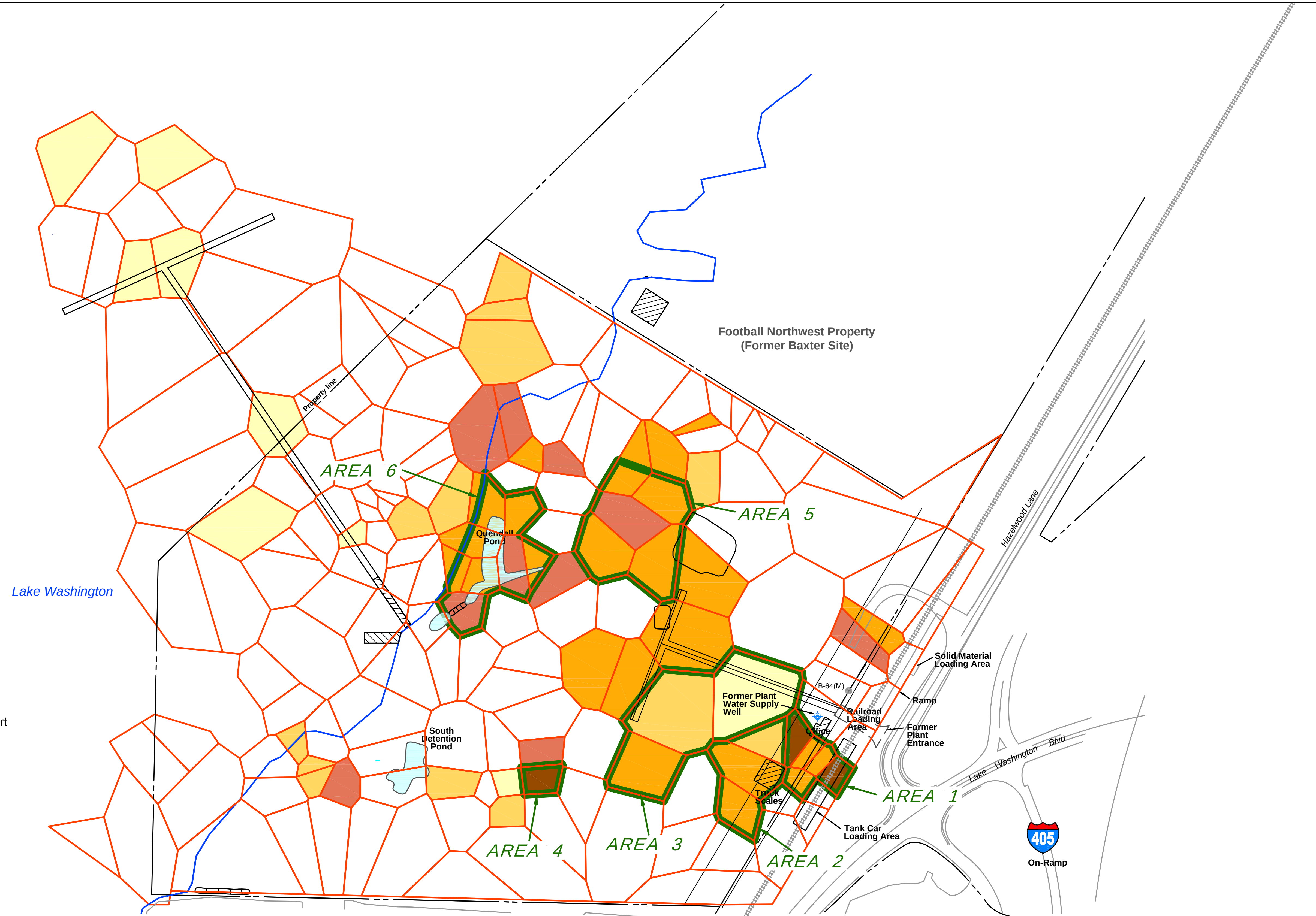
- DNAPL Thiessen Polygon
- DNAPL Treatment Area
- Detention Pond
- Existing Structure
- Historical Structure
- Other Historical Feature
- Current Shoreline

Maximum DNAPL Depth in Feet
Imported to model as depth of DNAPL constant concentration boundary condition (i.e., hydrocarbon source).

- 6.0
- 12.0
- 18.0
- 24.0
- 33.0

Notes

1. Modified from Figure 4.4-5 of Quendall Terminals RI Report (Anchor QEA and Aspect 2012).
2. Thiessen polygons based on midpoint between borings of adequate depth and characterization, truncated at property line.
3. See Appendix G, Figure G-1, and Table G-5 of the RI Report for maximum depth and area for each polygon.
4. DNAPL identified as oil-coated or oil-wetted soil. Sheen and stained soil not identified as DNAPL. See Section 4.3.1 for DNAPL definitions and Tables G-1 through G-4 in Appendix G of the RI Report for summaries of DNAPL characterization at each boring.
5. Refer to Table A-5 for area combinations evaluated.



N
0 200 400
Feet

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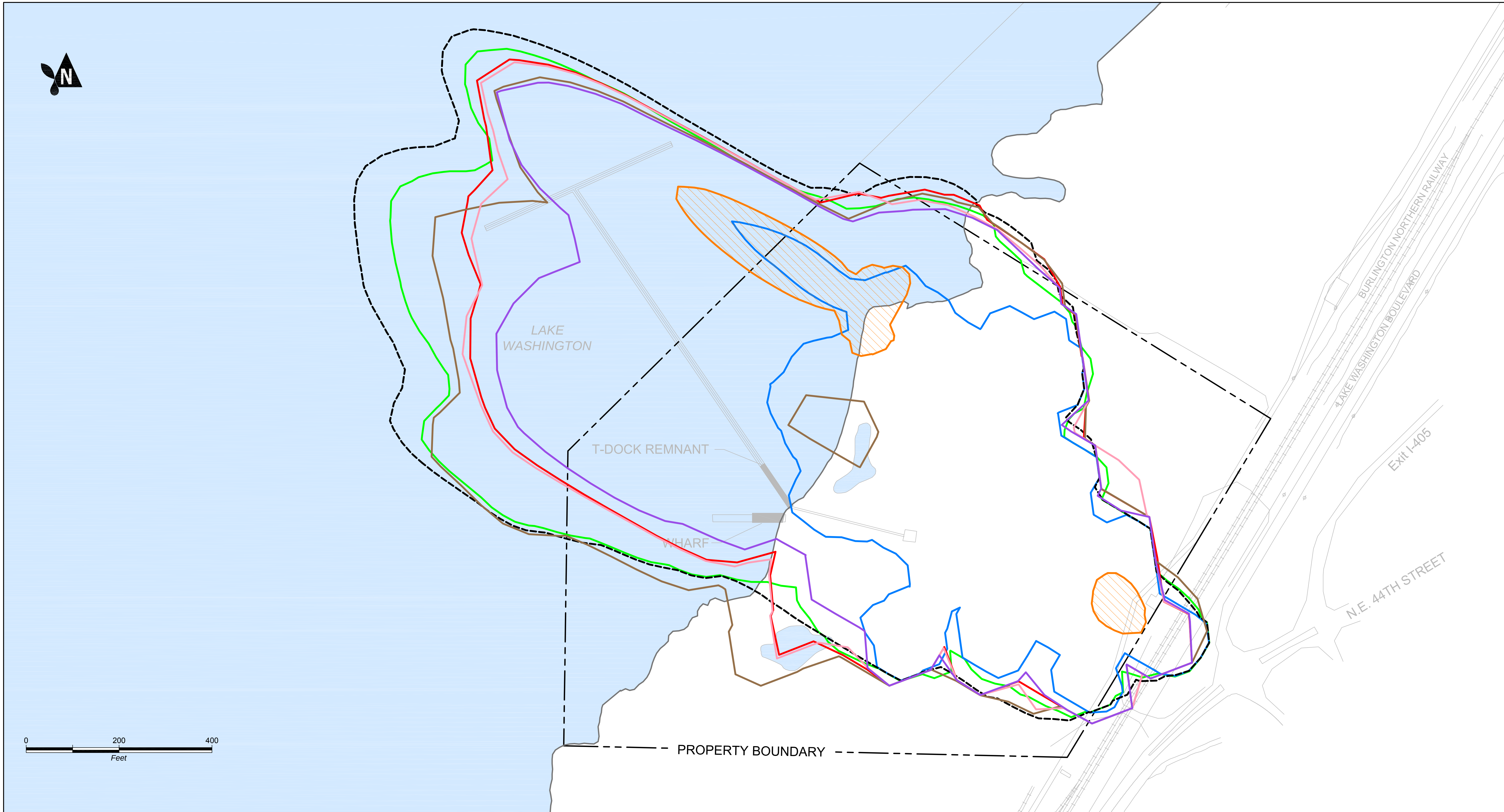
DNAPL Treatment Areas Evaluated for Alternative Development
Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS | FIRM: ASPECT | DRAWN BY: JJP/PMB/SCC | FIGURE NO. A-12

CAD Path: Q:\Quendall\020027 Quendall Terminals\2013-06 FS Draft Final\020027 A-2.dwg Figure A-2 (11 x 17) || Coordinate System: NAD 1983 State Plane Washington North FIPS 4601 Feet || Date Saved: Sep 13, 2013 11:43am || User: scudd



Plume Extent in Layer 2 *(Equal to 5 µg/L Isoconcentration Contour)*

- Alternative 1
- Alternative 2
- Alternative 3
- Alternative 4
- Alternative 5
- Alternative 6
- Alternative 7
- NA[1] Alternative 8
- Alternative 9
- NA[1] Alternative 10

Notes:

1. Alternatives 8 and 10 are not depicted because they result in no concentrations exceeding 5 µg/L.
2. Extents estimated by MODFLOW/MT3D. Alternative extents assume hydrocarbon source for 100 years, followed by implementation of alternative, and finally 100 years of attenuation.
3. Groundwater modeling results should be used as a relative tool for comparison and not considered a prediction of actual conditions. For example, model-estimated benzo[a]pyrene plume volumes for all alternatives are larger than anticipated based on current site data, due to simplifying modeling assumptions (see Appendix A, Section A3.2).

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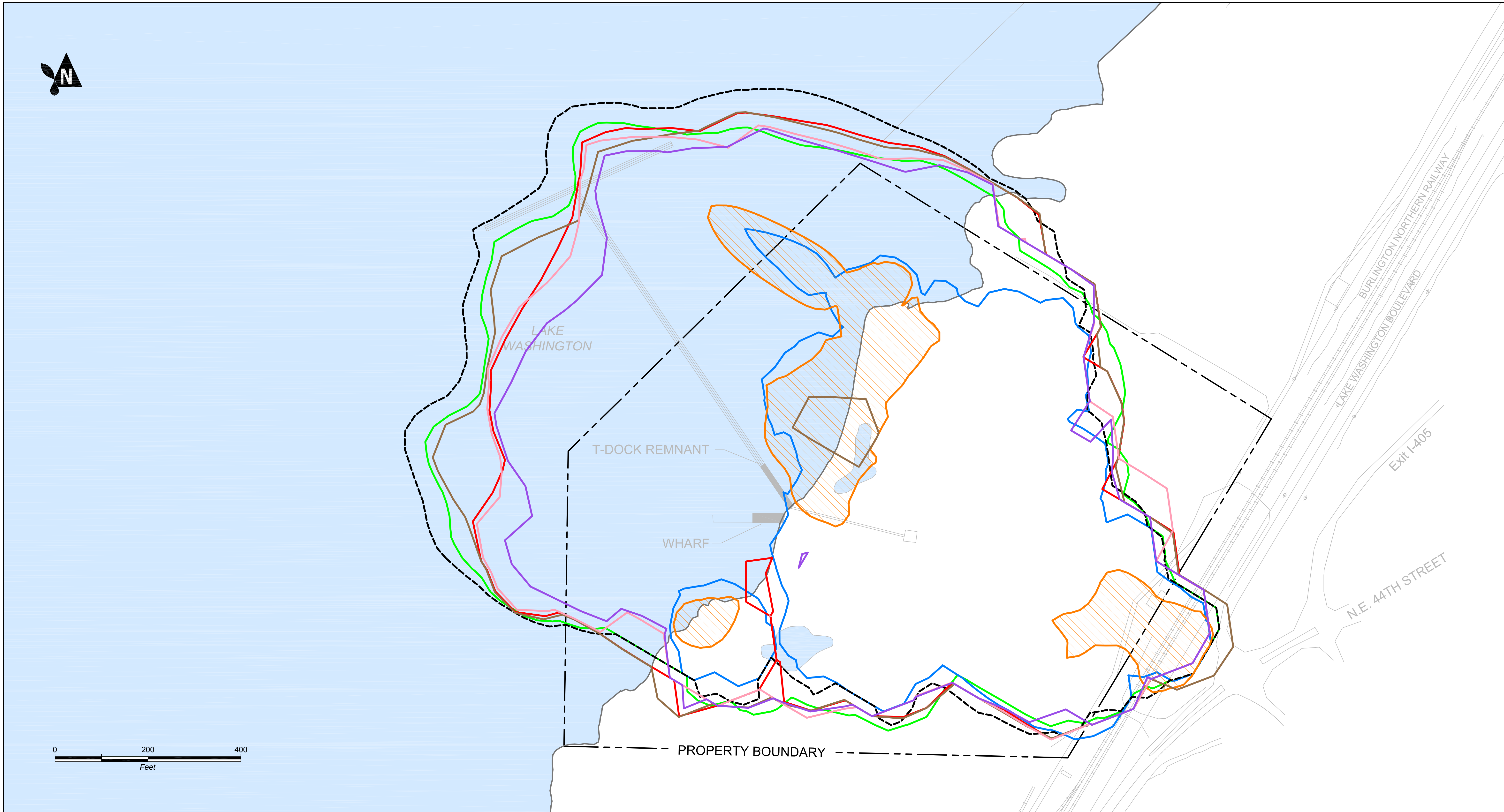
Model Simulated Benzene Plume
Plan View
Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS | FIRM: ASPECT | DRAWN BY: SM/SCC | FIGURE NO. A-13

CAD Path: Q:\Quendall\020027 Quendall Terminals\2013-06 FS Draft Final_Modeling Map Figures use ACAD 2012\020027-07.dwg Benzene || Coordinate System: NAD 1983 State Plane Washington North FIPS 4601 Feet || Date Saved: Sep 18, 2013 2:34pm || User: scudd



Plume Extent in Layer 2 *(Equal to 1.4 μg/L Isoconcentration Contour)*

- Alternative 1
- Alternative 2
- Alternative 3
- Alternative 4
- Alternative 5
- Alternative 6
- Alternative 7
- NA[1] Alternative 8
- Alternative 9
- NA[1] Alternative 10

Notes:

1. Alternatives 8 and 10 are not depicted because they result in no concentrations exceeding 1.4 μg/L.
2. Extents estimated by MODFLOW/MT3D. Alternative extents assume hydrocarbon source for 100 years, followed by implementation of alternative, and finally 100 years of attenuation.
3. Layer 2 is the shallowest active layer and approximates the maximum plume extent for most alternatives. Alternative 9, however, produces a plume extent that is greater in deeper layers (see Figure A-19).
4. Groundwater modeling results should be used as a relative tool for comparison and not considered a prediction of actual conditions. For example, model-estimated benzo[a]pyrene plume volumes for all alternatives are larger than anticipated based on current site data, due to simplifying modeling assumptions (see Appendix A, Section A3.2).

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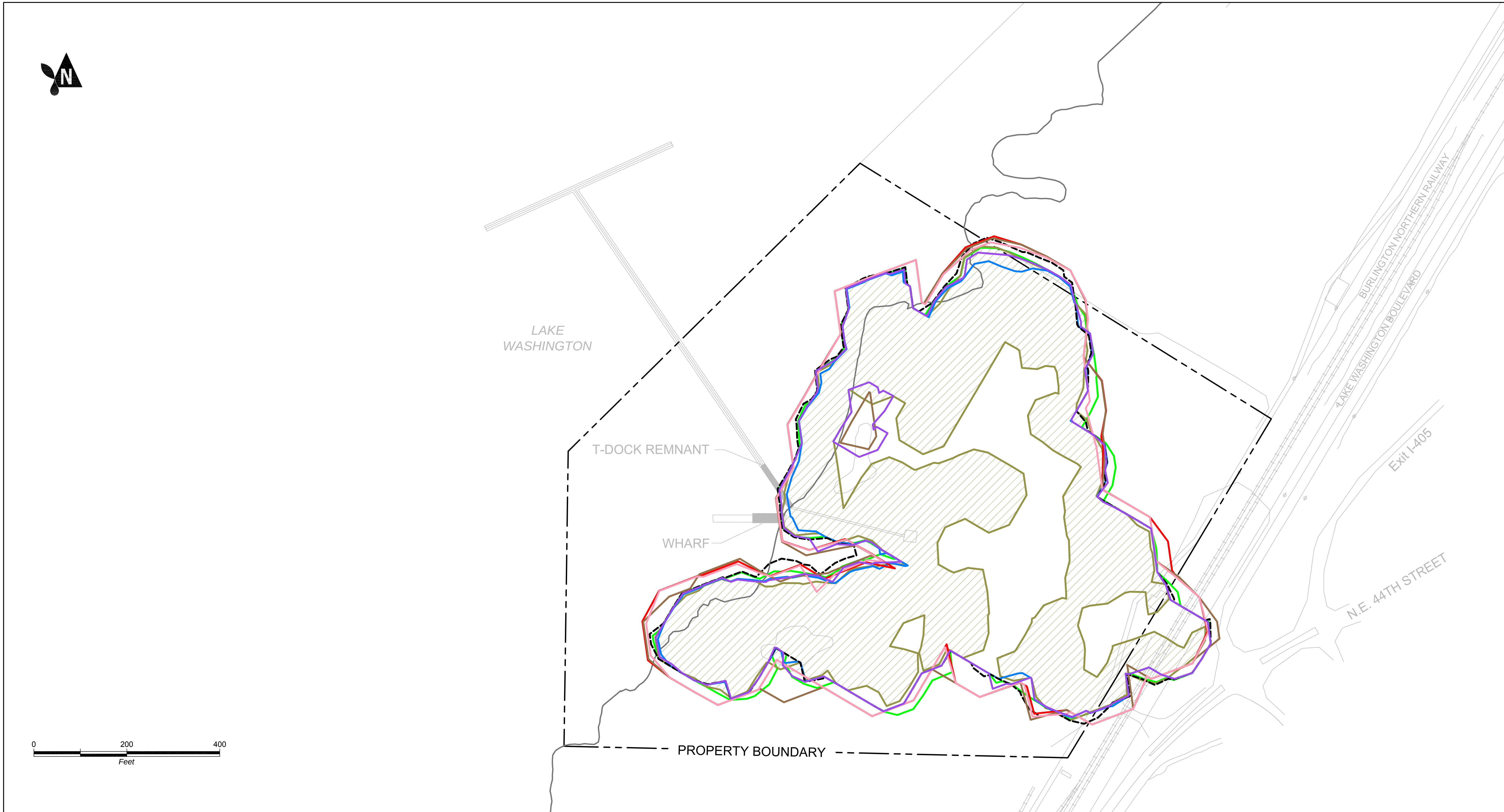
Model Simulated Naphthalene Plume Plan View

Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS | FIRM: ASPECT | DRAWN BY: SM/SCC | FIGURE NO. A-14

CAD Path: Q:\Quendall\020027 Quendall Terminals\2013-06 FS Draft Final_Modeling Map Figures use ACAD 2012\020027-07.dwg Naphthalene || Coordinate System: NAD 1983 State Plane Washington North FIPS 4601 Feet || Date Saved: Sep 18, 2013 2:39pm || User: scudd



Plume Extent in Layer 2 *(Equal to 0.2 µg/L Isoconcentration Contour)*

- Alternative 1
- Alternative 2
- Alternative 3
- Alternative 4
- Alternative 5
- Alternative 6
- Alternative 7
- Alternative 8
- NA² Alternative 9
- NA³ Alternative 10

Note:

1. Extents estimated by MODFLOW/MT3D. Alternative extents assume hydrocarbon source for 100 years, followed by implementation of alternative, and finally 100 years of attenuation.
2. No exceedances predicted in layer 2 for Alternative 9; however, exceedances are predicted in deeper layers (see figure A-20).
3. Modeling results do not include the potential contribution of residuals resulting from removal actions (i.e., excavation or dredging). It is expected, based on a model sensitivity analysis (see Appendix A, Section A5.1.2.2), that residuals will result in benzo[a]pyrene exceedances after 100 years for all alternatives, including Alternative 10.
4. Groundwater modeling results should be used as a relative tool for comparison and not considered a prediction of actual conditions. For example, model-estimated benzo[a]pyrene plume volumes for all alternatives are larger than anticipated based on current site data, due to simplifying modeling assumptions (see Appendix A, Section A3.2).

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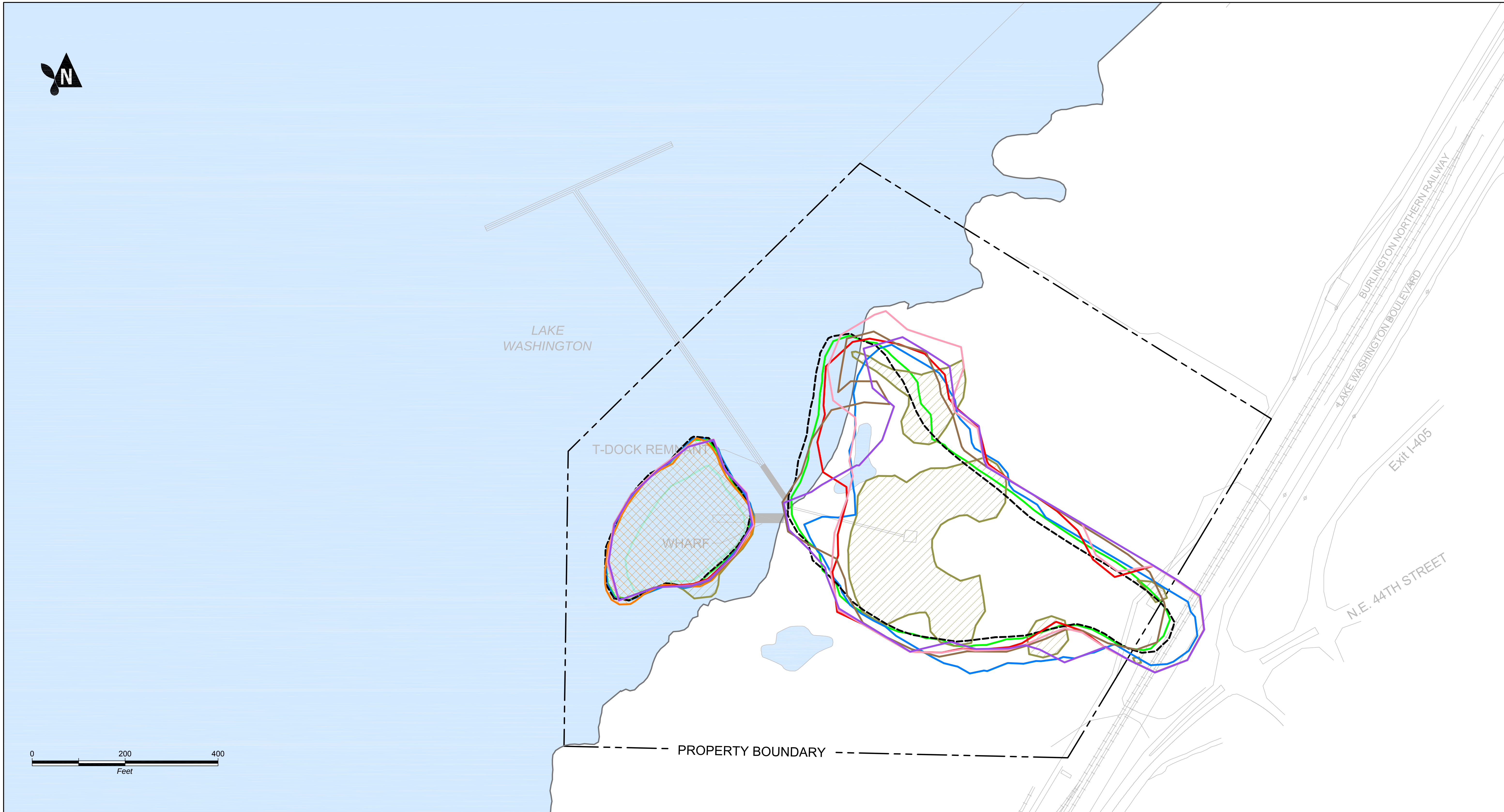
Model Simulated Benzo(a)pyrene Plume Plan View

Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS

FIRM: ASPECT
DRAWN BY: SM/SCC
FIGURE NO. A-15

CAD Path: Q:\Quendall\020027 Quendall Terminals\2013-06 FS Draft Final_Modeling Map Figures use ACAD 2012\020027-07.dwg Benzo-a-Pyrene || Coordinate System: NAD 1983 State Plane Washington North FIPS 4601 Feet || Date Saved: Sep 18, 2013 4:31pm || User: scudd



Plume Extent in Layer 2 *(Equal to 10 µg/L Isoconcentration Contour)*

- Alternative 1
- Alternative 2
- Alternative 3
- Alternative 4
- Alternative 5
- Alternative 6
- Alternative 7
- Alternative 8
- Alternative 9
- Alternative 10

Notes:

1. Extents estimated by MODFLOW/MT3D. Alternative extents assume 100 years of attenuation following implementation of alternative.
2. Layer 2 is the shallowest active layer and approximates the maximum plume extent for most alternatives. Alternatives 8, 9, and 10, however, produce plume extents that are greater in deeper layers (see Figure A-21).
3. Groundwater modeling results should be used as a relative tool for comparison and not considered a prediction of actual conditions. For example, model-estimated benzo[a]pyrene plume volumes for all alternatives are larger than anticipated based on current site data, due to simplifying modeling assumptions (see Appendix A, Section A3.2).

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Model Simulated Arsenic Plume Plan View

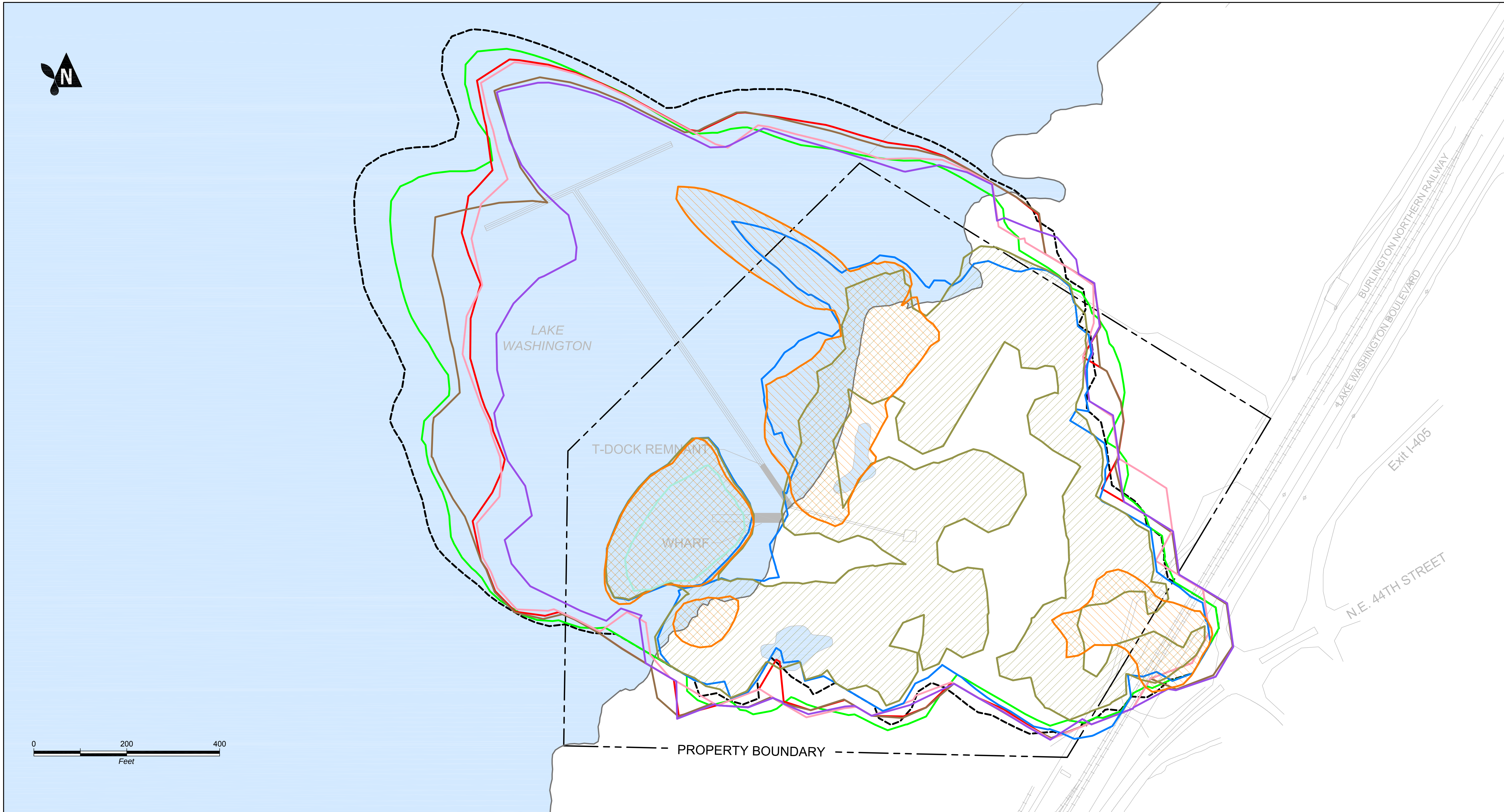
Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS

FIRM: ASPECT
DRAWN BY: SM/SCC

FIGURE NO. **A-16**

CAD Path: Q:\Quendall\020027 Quendall Terminals\2013-06 FS Draft Final_Modeling Map Figures use ACAD 2012\020027-07.dwg Arsenic || Coordinate System: NAD 1983 State Plane Washington North FIPS 4601 Feet || Date Saved: Sep 18, 2013 2:51pm || User: scudd



Plume Extent in Layer 2 *(includes groundwater that exceeds any groundwater PRGs²)*

- Alternative 1
- Alternative 2
- Alternative 3
- Alternative 4
- Alternative 5
- Alternative 6
- Alternative 7
- Alternative 8
- Alternative 9
- Alternative 10

Notes:

1. Extents estimated by MODFLOW/MT3D. Alternative extents simulate 100 years of attenuation following implementation of alternative.
2. Groundwater PRGs : *Napthalene 1.4 µg/L*
   Arsenic 10 µg/L
   Benzene 5 µg/L
   Benzo(a)Pyrene 0.2 µg/L
3. Layer 2 is the shallowest active layer and approximates the maximum plume extent for most alternatives. Alternatives 9 and 10, however, produces a plume extent that is greater in deeper layers.
4. Groundwater modeling results should be used as a relative tool for comparison and not considered a prediction of actual conditions. For example, model-estimated benzo[a]pyrene plume volumes for all alternatives are larger than anticipated based on current site data, due to simplifying modeling assumptions (see Appendix A, Section A3.2).

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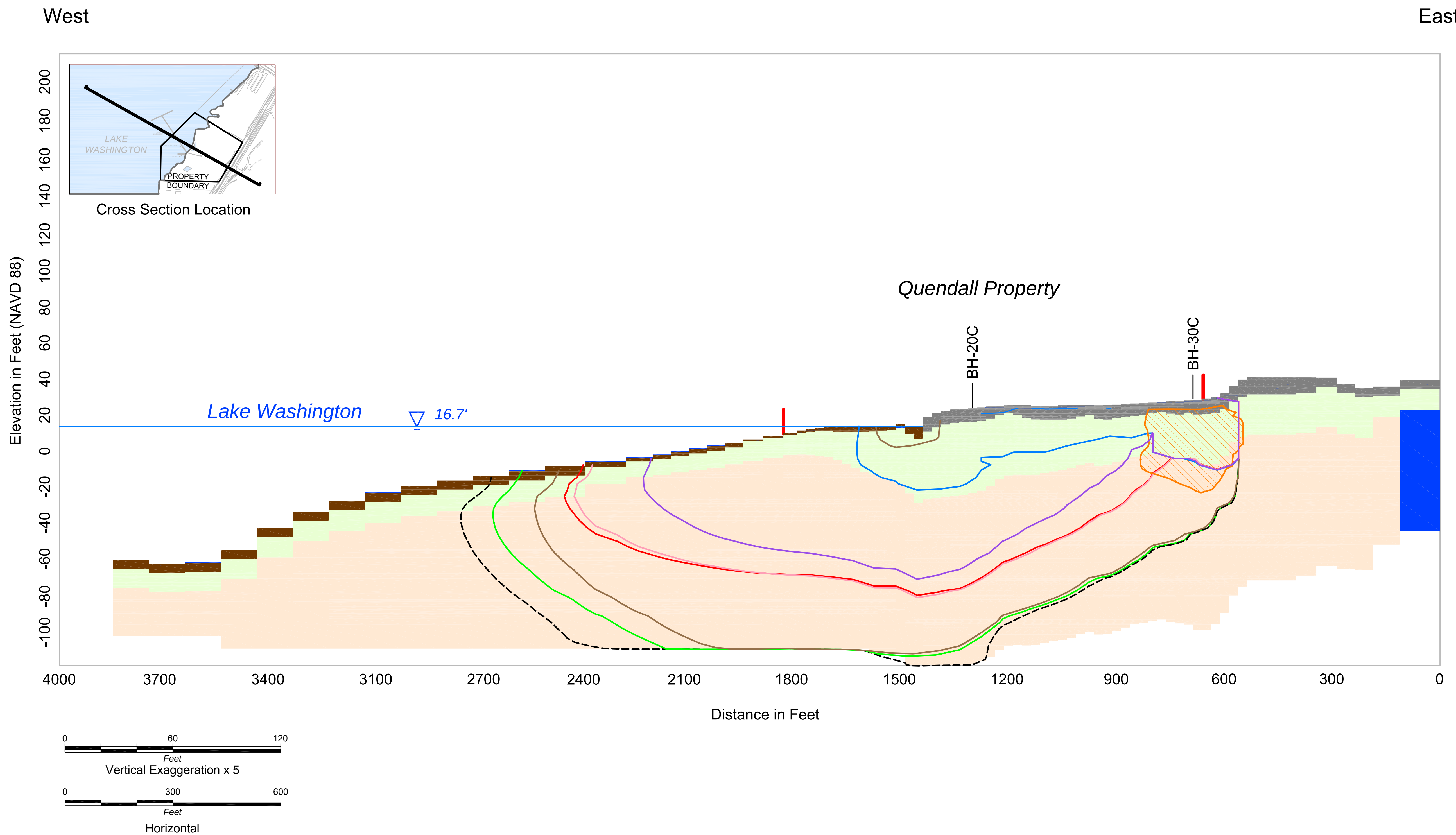
Model Simulated Aggregate Plume
Plan View

Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS

FIRM: ASPECT
DRAWN BY: SM/SCC

FIGURE NO.
A-17



All elevations are in feet NAVD 88.

Legend

- Fill
- Shallow Alluvium
- Deeper Alluvium
- Lake Washington Sediments
- Constant Head Boundary Cell
- Quendall Property Boundary

Plume Extent in Column 76 *(Equal to 5 µg/L Isoconcentration Contour)*

- Alternative 1
- Alternative 2
- Alternative 3
- Alternative 4
- Alternative 5
- Alternative 6
- Alternative 7
- NA[1] Alternative 8
- Alternative 9
- NA[1] Alternative 10

Notes:

1. Alternatives 8 and 10 are not depicted because they result in no concentrations exceeding 5 µg/L.
2. Extents estimated by MODFLOW/MT3D. Alternative extents assume hydrocarbon source for 100 years, followed by implementation of alternative, and finally 100 years of attenuation.
3. Groundwater modeling results should be used as a relative tool for comparison and not considered a prediction of actual conditions. For example, model-estimated benzo[a]pyrene plume volumes for all alternatives are larger than anticipated based on current site data, due to simplifying modeling assumptions (see Appendix A, Section A3.2).

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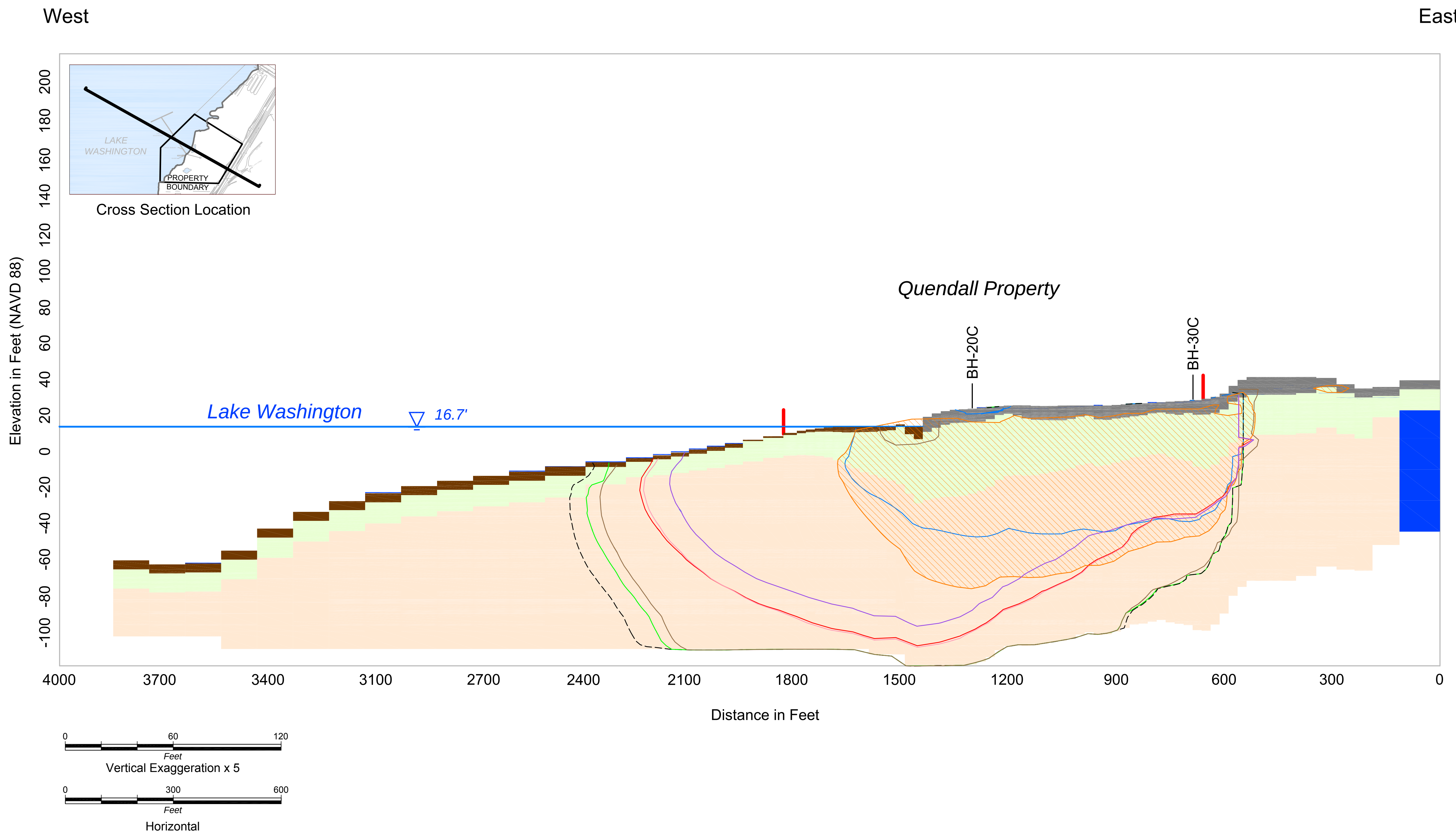
Model Simulated Benzene Plume Cross Section View

Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS | FIRM: ASPECT | DRAWN BY: SM/SCC | FIGURE NO. A-18



All elevations are in feet NAVD 88.

Legend

Fill

Shallow Alluvium

Deeper Alluvium

Lake Washington Sediments

Constant Head Boundary Cell

Quendall Property Boundary

Plume Extent in Column 76 *(Equal to 1.4 µg/L Isoconcentration Contour)*

- - - -	Alternative 1		Alternative 7
	Alternative 2	NA[1]	Alternative 8
	Alternative 3		Alternative 9
	Alternative 4	NA[1]	Alternative 10
	Alternative 5		
	Alternative 6		

Notes:

1. Alternatives 8 and 10 are not depicted because they result in no concentrations exceeding 1.4 µg/L.
2. Extents estimated by MODFLOW/MT3D. Alternative extents assume hydrocarbon source for 100 years, followed by implementation of alternative, and finally 100 years of attenuation.
3. Groundwater modeling results should be used as a relative tool for comparison and not considered a prediction of actual conditions. For example, model-estimated benzo[a]pyrene plume volumes for all alternatives are larger than anticipated based on current site data, due to simplifying modeling assumptions (see Appendix A, Section A3.2).

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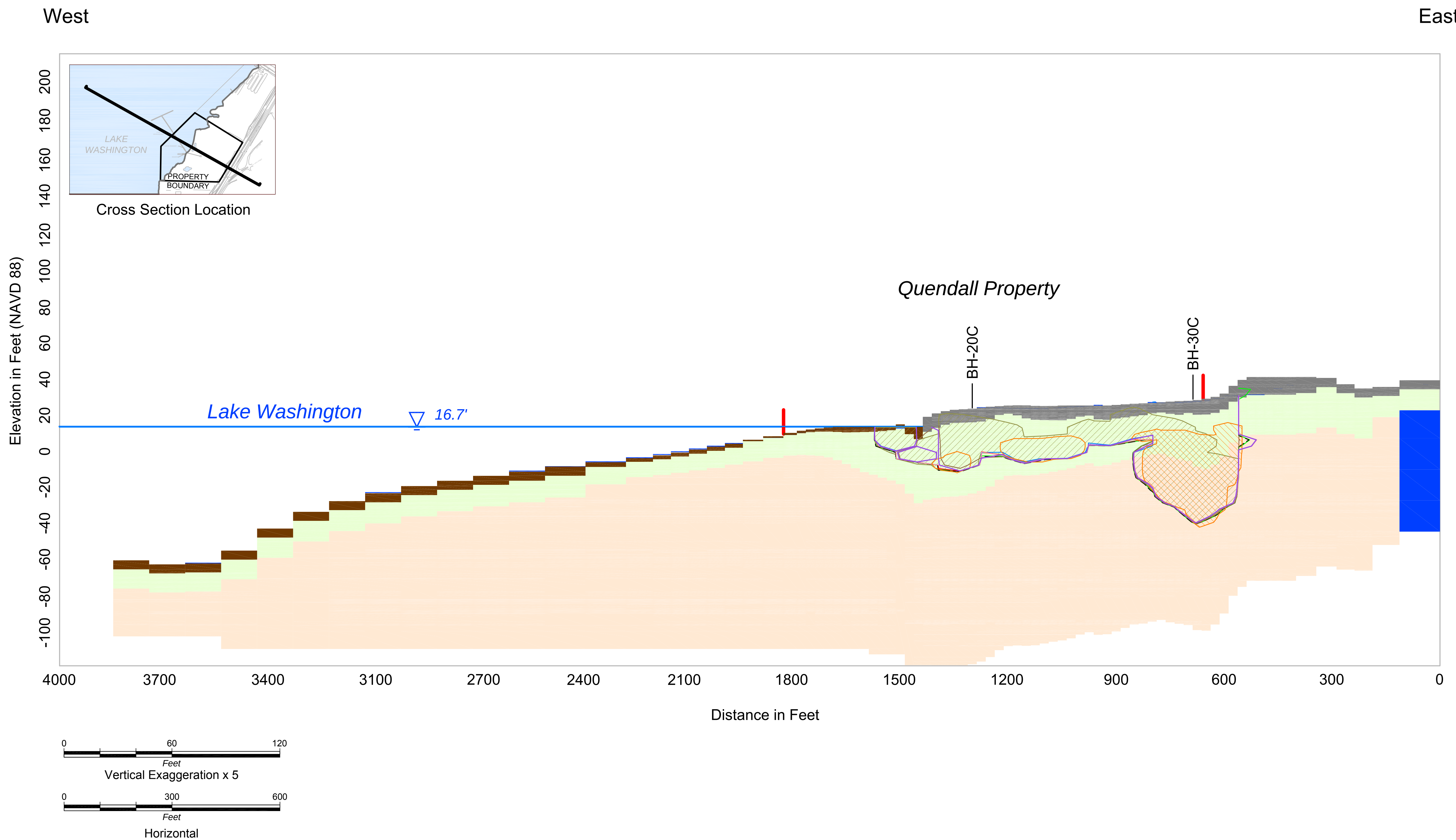
Model Simulated Naphthalene Plume Cross Section View

Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS | FIRM: ASPECT | DRAWN BY: SM/SCC | FIGURE NO. A-19

CAD Path: Q:\Quendall\020027 Quendall Terminals\2013-06 FS Draft Final\020027-10 Naphthalene.dwg Naphthalene || Coordinate System: NAD 1983 State Plane Washington North FIPS 4601 Feet || Date Saved: Sep 19, 2013 10:58am || User: scudd



All elevations are in feet NAVD 88.

Legend

- Fill
- Shallow Alluvium
- Deeper Alluvium
- Lake Washington Sediments
- Constant Head Boundary Cell
- Quendall Property Boundary

Plume Extent in Column 76 *(Equal to 0.2 µg/L Isoconcentration Contour)*

Alternative 1	Alternative 6
Alternative 2	Alternative 7
Alternative 3	Alternative 8
Alternative 4	Alternative 9
Alternative 5	NA² Alternative 10

Note:

1. Extents estimated by MODFLOW/MT3D. Alternative extents assume hydrocarbon source for 100 years, followed by implementation of alternative, and finally 100 years of attenuation.
2. Modeling results do not include the potential contribution of residuals resulting from removal actions (i.e., excavation or dredging). It is expected, based on a model sensitivity analysis (see Appendix A, Section A5.1.2.2), that residuals will result in benzo[a]pyrene exceedances after 100 years for all alternatives, including Alternative 10.
3. Groundwater modeling results should be used as a relative tool for comparison and not considered a prediction of actual conditions. For example, model-estimated benzo[a]pyrene plume volumes for all alternatives are larger than anticipated based on current site data, due to simplifying modeling assumptions (see Appendix A, Section A3.2).

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Model Simulated Benzo(a)pyrene Plume Cross Section View

Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS

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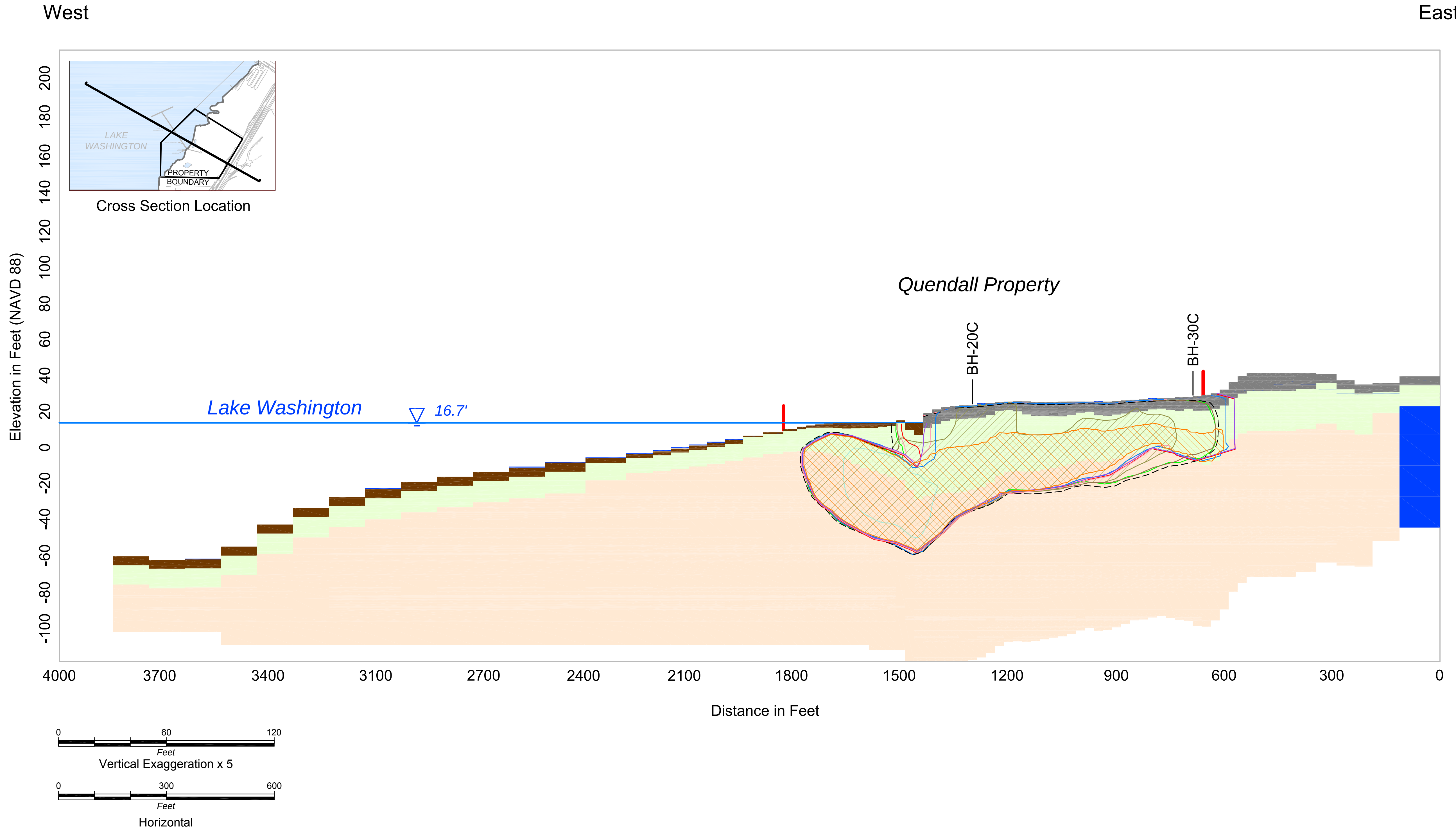
DRAWN BY: SM/SCC

FIGURE NO. **A-20**



All elevations are in feet NAVD 88.

Legend

- Fill
- Shallow Alluvium
- Deeper Alluvium
- Lake Washington Sediments
- Constant Head Boundary Cell
- Quendall Property Boundary

Plume Extent in Column 76 *(Equal to 10 µg/L Isoconcentration Contour)*

- Alternative 1
- Alternative 2
- Alternative 3
- Alternative 4
- Alternative 5
- Alternative 6
- Alternative 7
- Alternative 8
- Alternative 9
- Alternative 10

Notes:

1. Extents estimated by MODFLOW/MT3D. Alternative extents assume 100 years of attenuation following implementation of alternative.
2. Groundwater modeling results should be used as a relative tool for comparison and not considered a prediction of actual conditions. For example, model-estimated benzo[a]pyrene plume volumes for all alternatives are larger than anticipated based on current site data, due to simplifying modeling assumptions (see Appendix A, Section A3.2).

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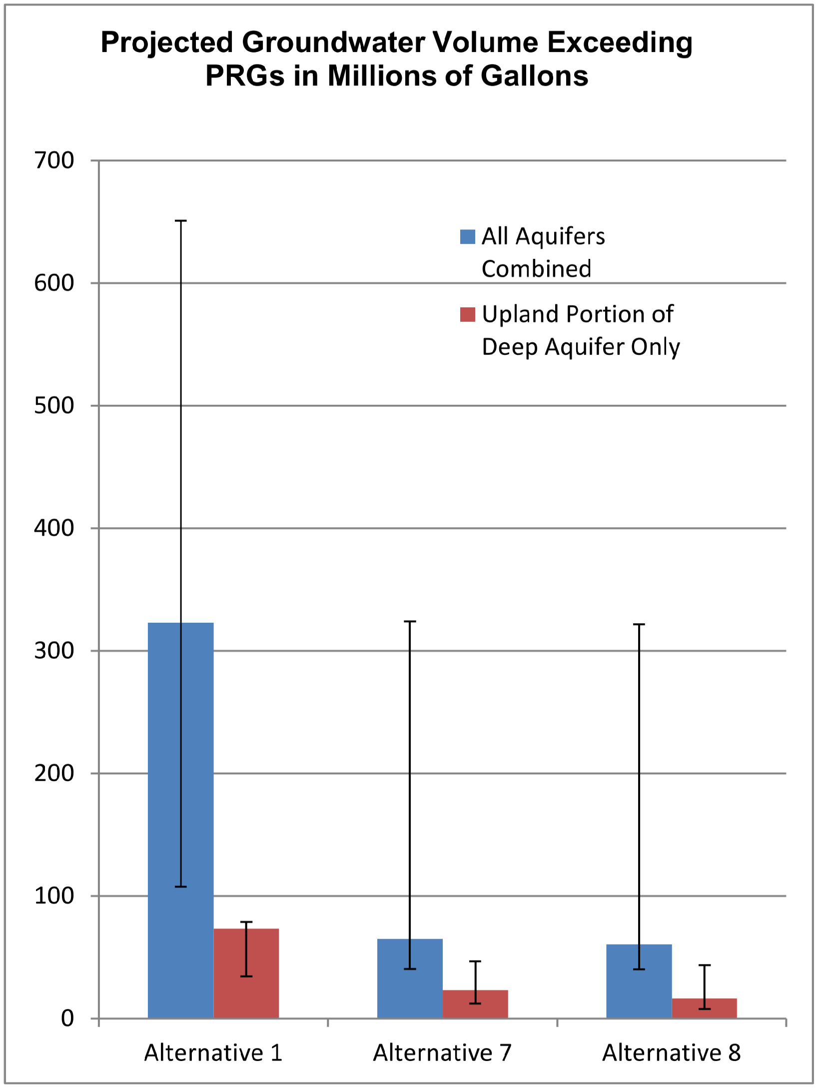
Model Simulated Arsenic Plume
Cross Section View

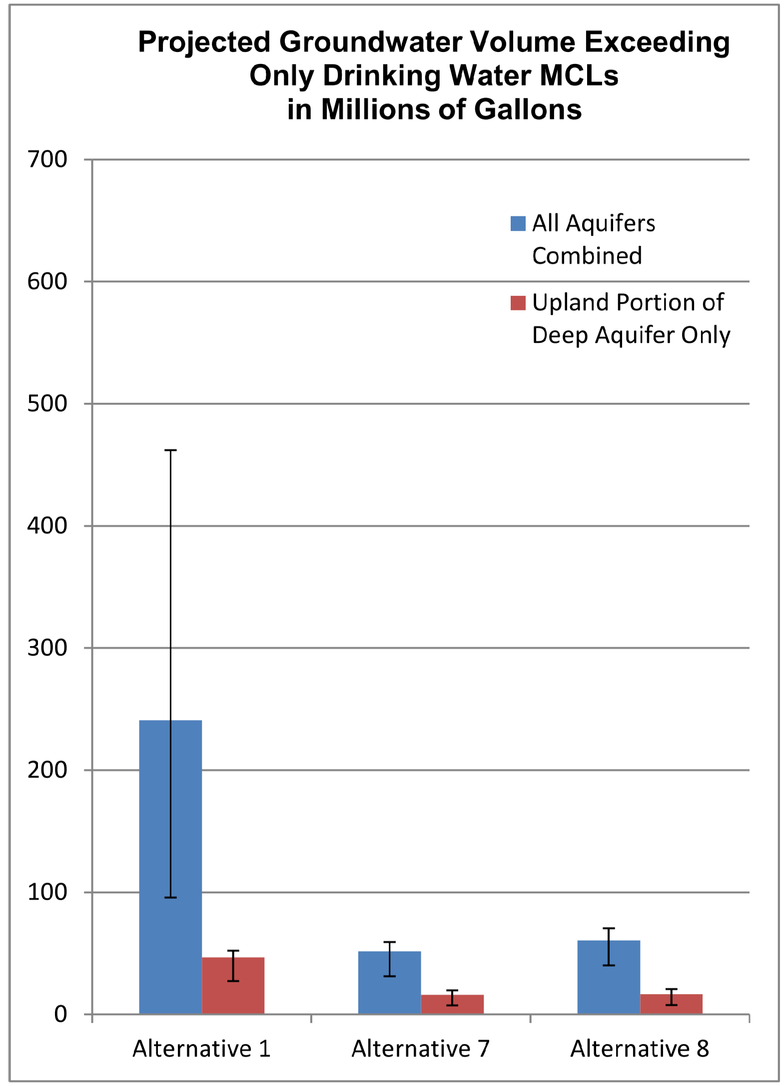
Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS

FIRM: ASPECT
DRAWN BY: SM/SCC
FIGURE NO. A-21

CAD Path: Q:\Quendall\020027 Quendall Terminals\2013-06 FS Draft Final\020027-09 Arsenic.dwg Arsenic || Coordinate System: NAD 1983 State Plane Washington North FIPS 4601 Feet || Date Saved: Sep 23, 2013 1:23pm || User: scudd





Note:
1. Error bar represents range between best and worst cases.

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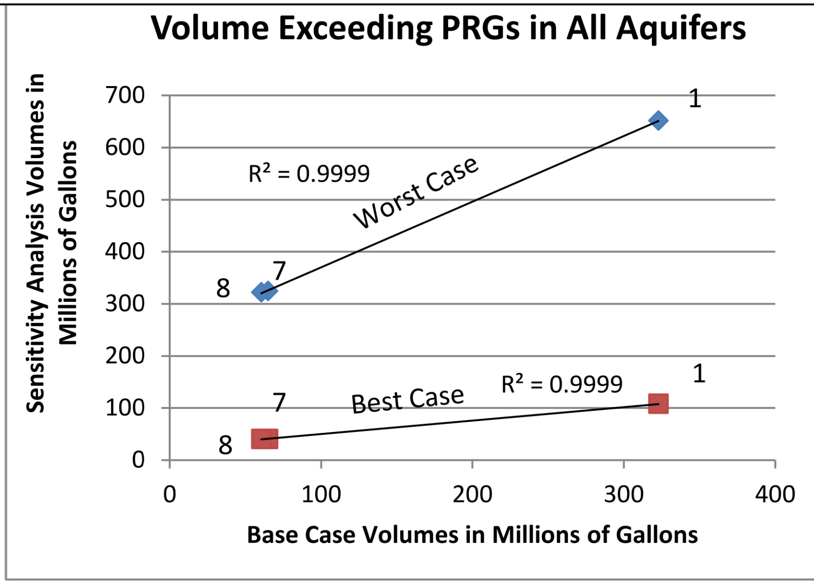
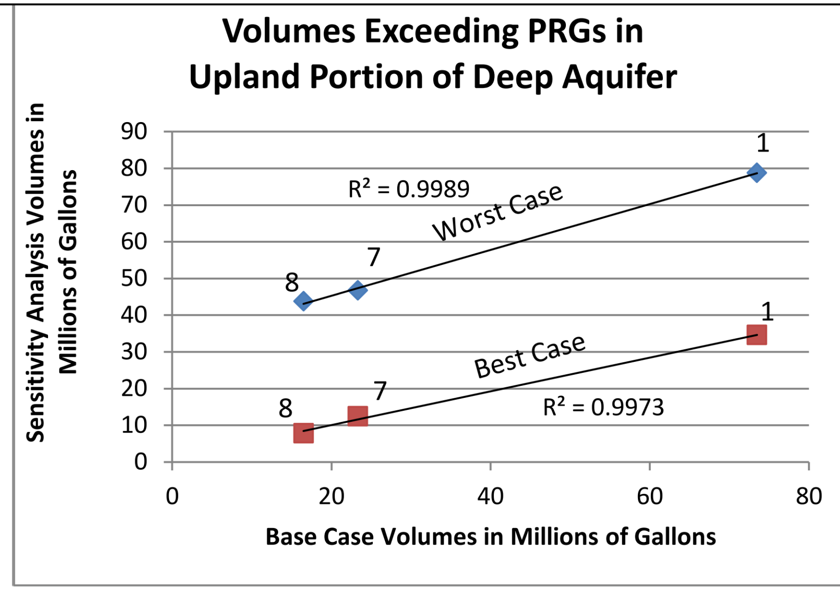
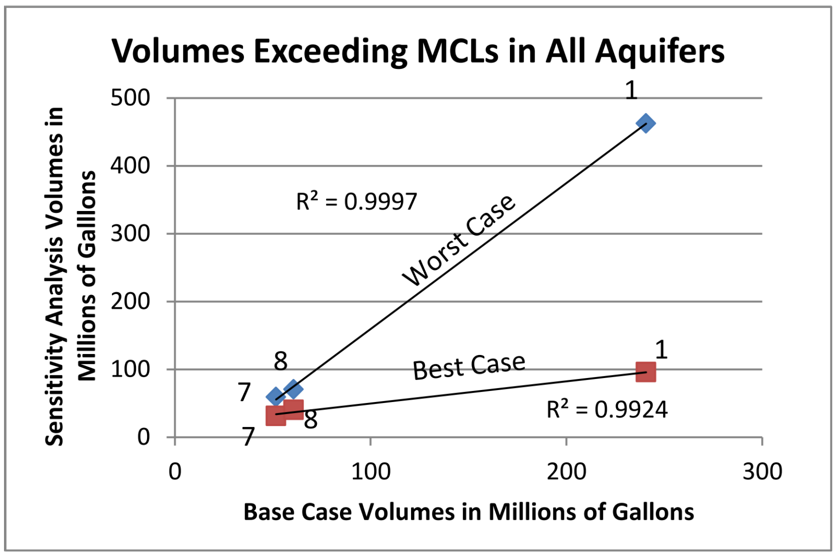
October 14, 2013

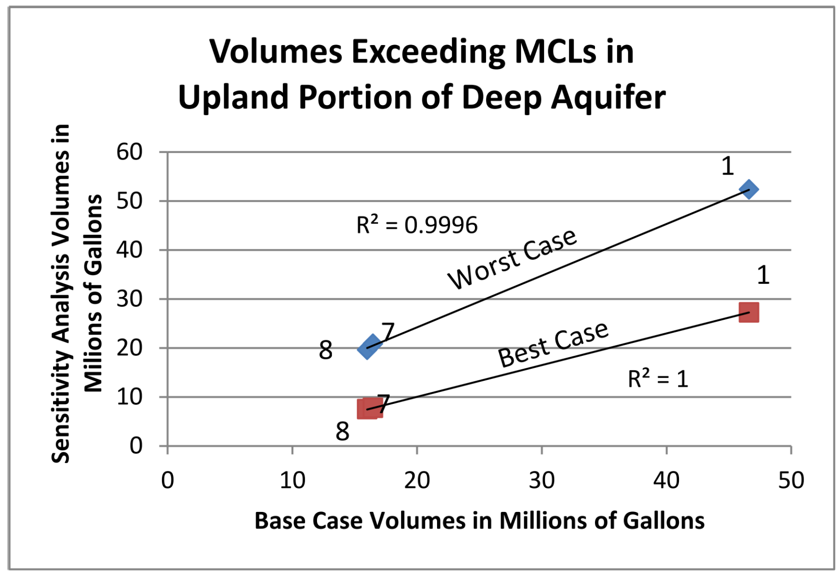
Sensitivity Analysis Results
Aggregate Plume Volume
Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS | FIRM: ASPECT | DRAWN BY: JJP/SCC | FIGURE NO. A-22









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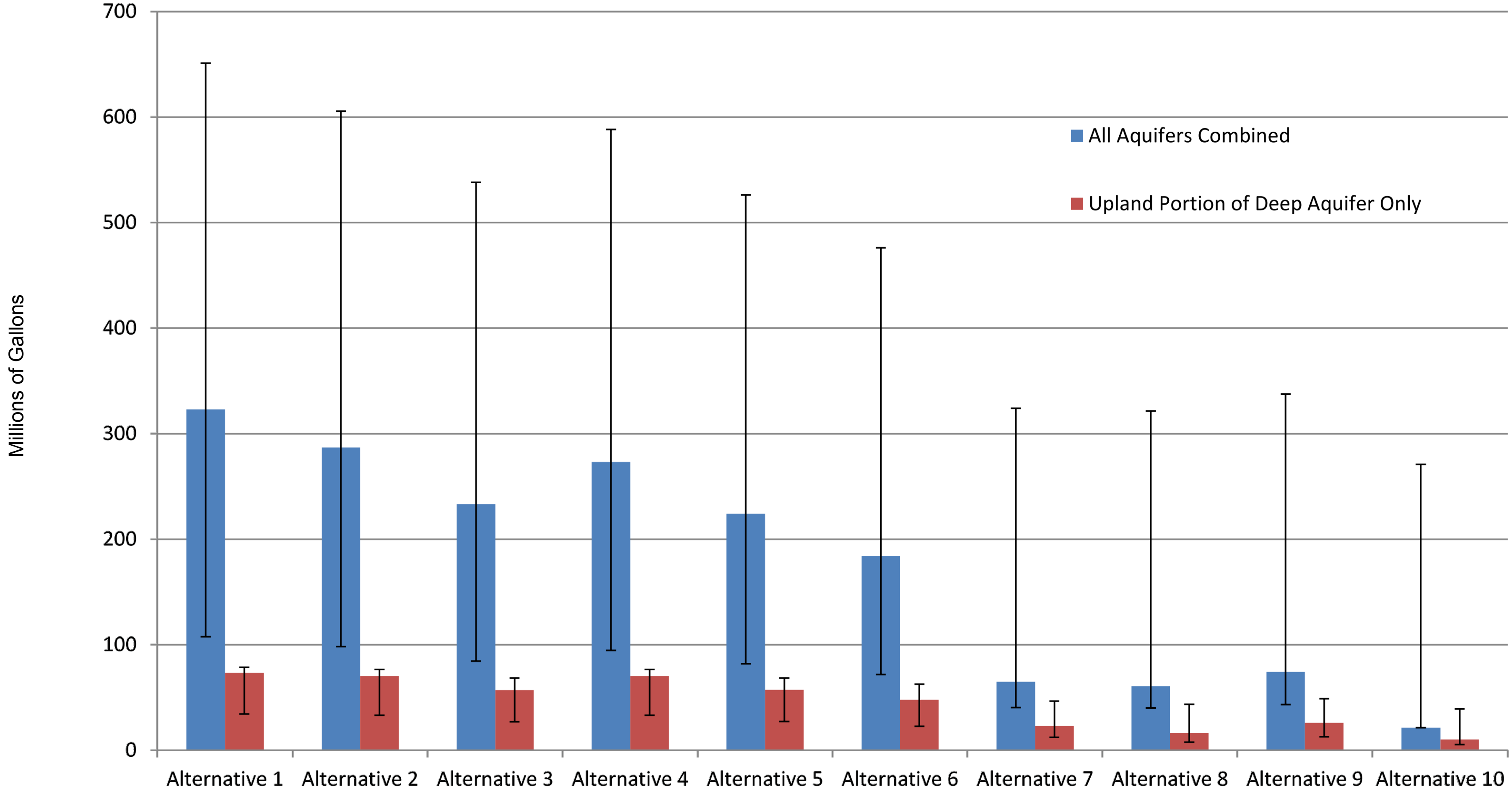
Linear Interpolation of Sensitivity Analysis Results-Aggregate Plume Volume

Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS | FIRM: ASPECT DRAWN BY: JJP/SCC | FIGURE NO. A-23



Notes:

1. Groundwater modeling results should be used as a relative tool for comparison and not considered a prediction of actual conditions. For example, model-estimated benzo[a]pyrene volumes for all alternatives are larger than anticipated based on current site data, due to simplifying modeling assumptions (see Appendix A, Section A3.2).
2. Modeling results do not include the potential contribution of residuals resulting from removal actions (i.e., excavation or dredging). It is expected, based on a model sensitivity analysis (see Appendix A, Section A5.1.2), that residuals will result in benzo[a]pyrene exceedances after 100 years for all alternatives, including Alternative 10.
3. Error bar represents range between best and worst cases.

CAD Path: Q:\Quendall\020027 Quendall Terminals\2013-06 FS Draft Final\020027-A-22 thru A-34.dwg A-24 || Date Saved: Oct 10, 2013 12:28pm || User: scudd

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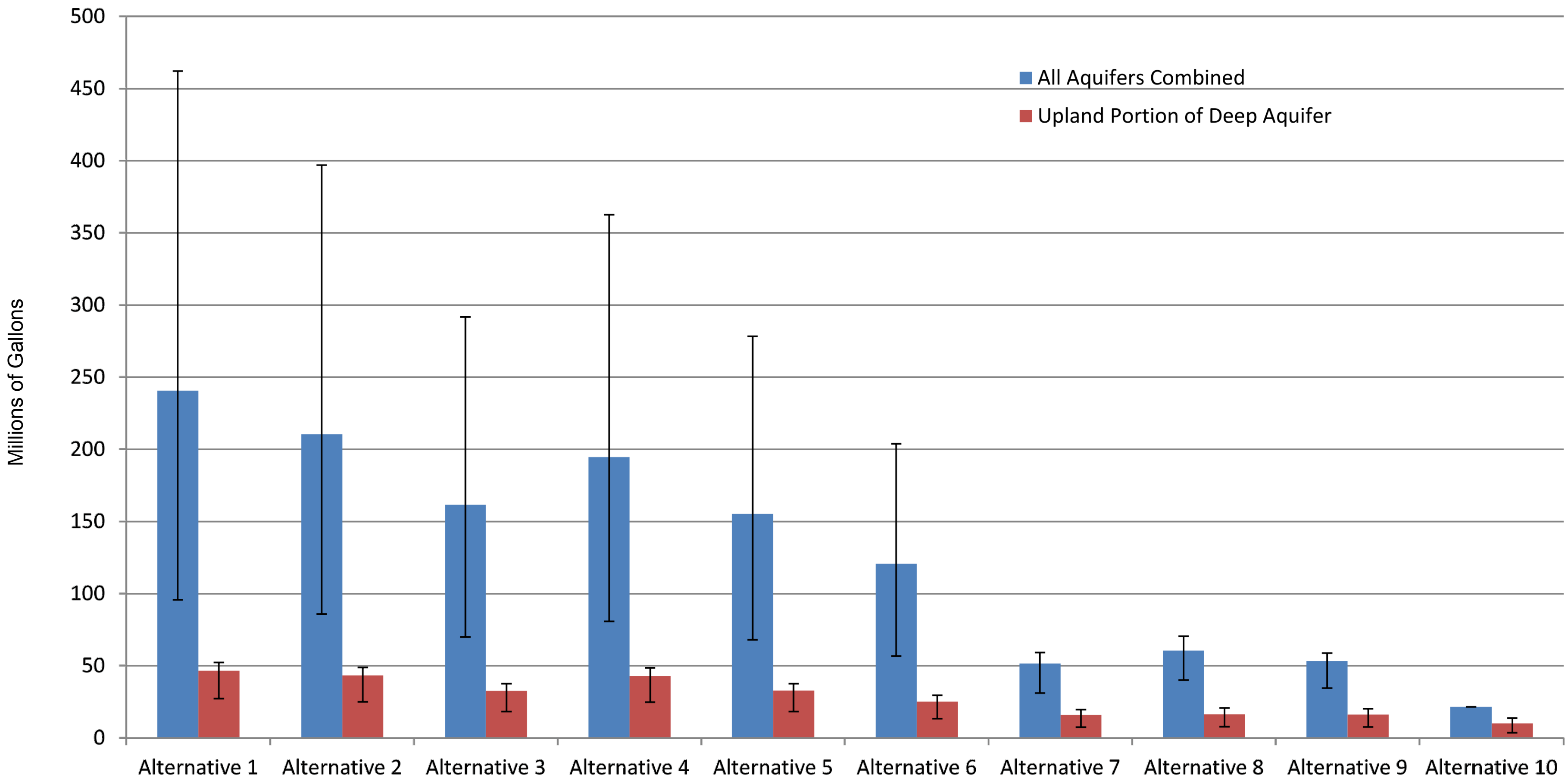
October 14, 2013

Estimated Sensitivity Analysis Results
Aggregate Plume Volume
Exceeding Drinking Water PRGs

Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS | FIRM: ASPECT | DRAWN BY: JJP/SCC | FIGURE NO. A-24



Notes:

1. Groundwater modeling results should be used as a relative tool for comparison and not considered a prediction of actual conditions. For example, model-estimated benzo[a]pyrene volumes for all alternatives are larger than anticipated based on current site data, due to simplifying modeling assumptions (see Appendix A, Section A3.2).
2. Modeling results do not include the potential contribution of residuals resulting from removal actions (i.e., excavation or dredging). It is expected, based on a model sensitivity analysis (see Appendix A, Section A5.1.2), that residuals will result in benzo[a]pyrene exceedances after 100 years for all alternatives, including Alternative 10.
3. Error bar represents range between best and worst cases.

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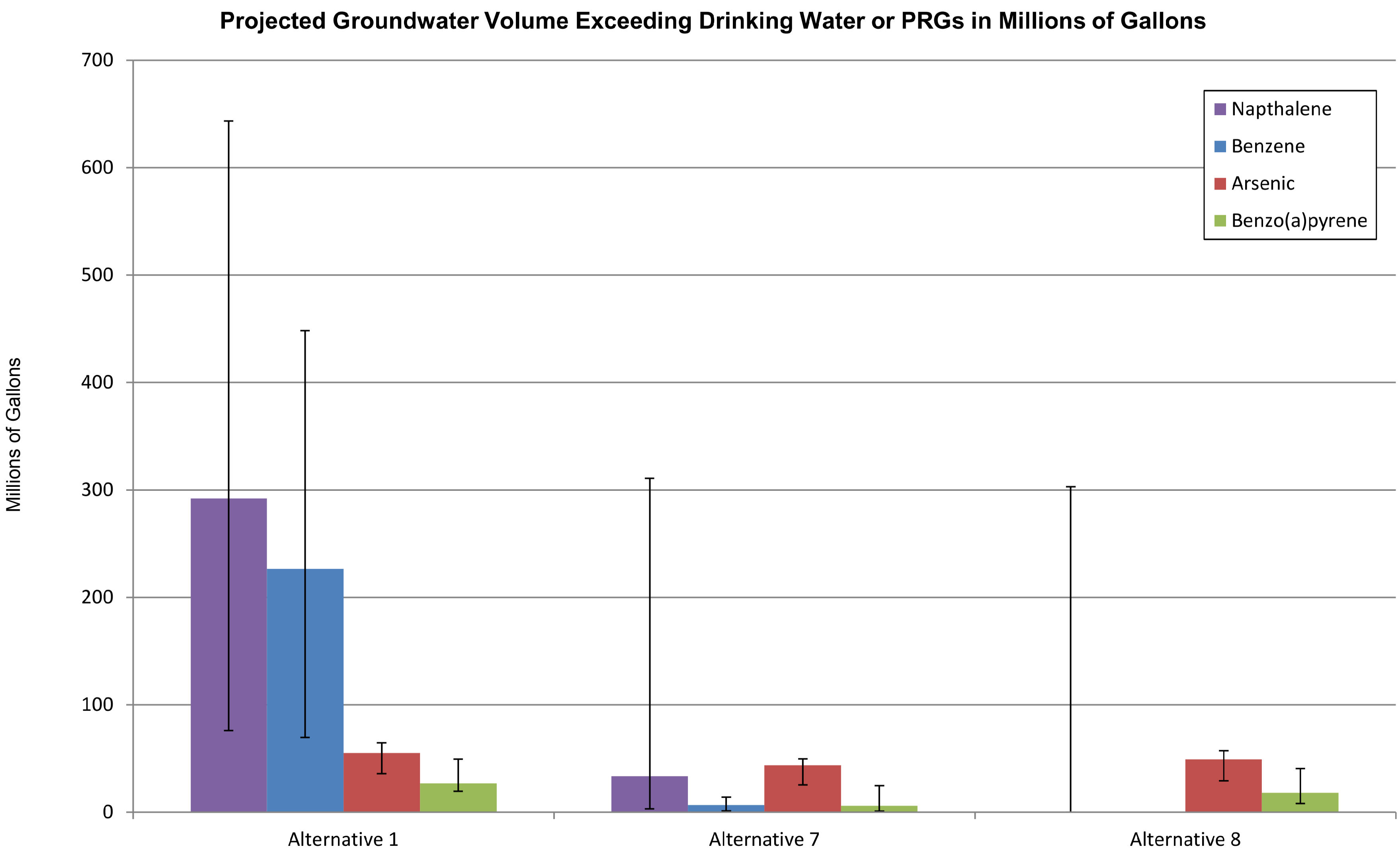
October 14, 2013

Estimated Sensitivity Analysis Results
Aggregate Plume Volume
Exceeding Drinking Water MCLs Only

Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS | FIRM: ASPECT | DRAWN BY: JJP/SCC | FIGURE NO. **A-25**



Note:
1. Error bar represents range between best and worst cases.

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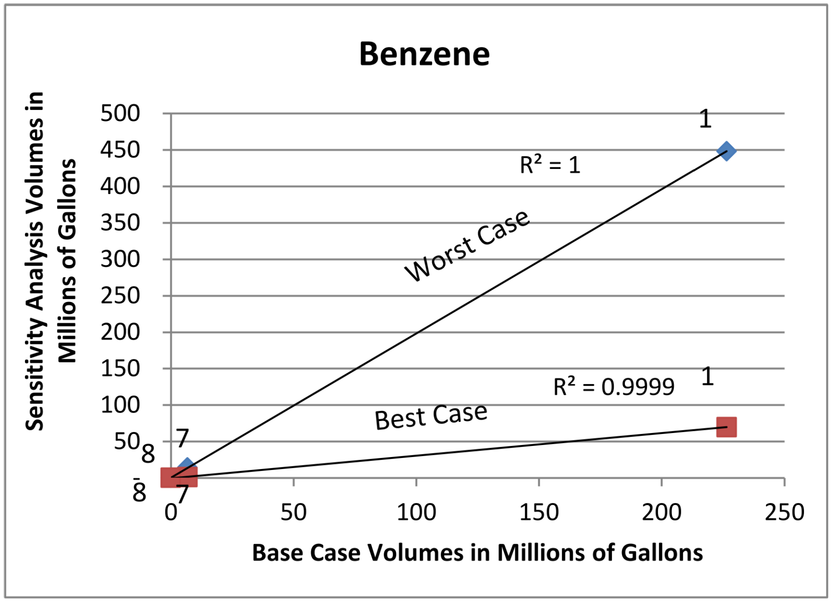
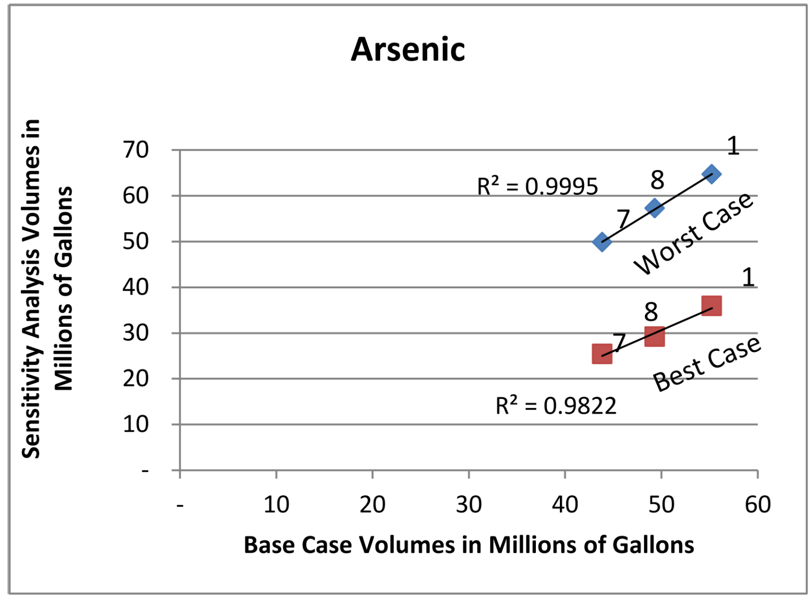
October 14, 2013

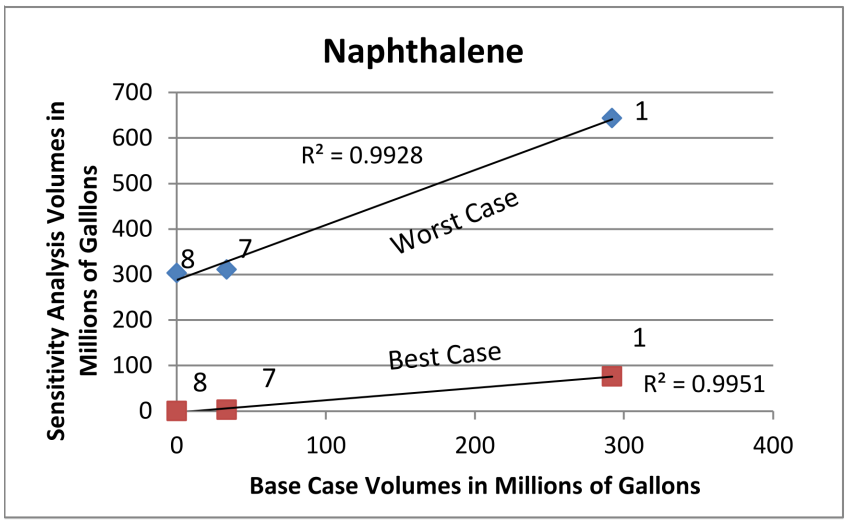
Sensitivity Analysis Results by COC Plume Volume

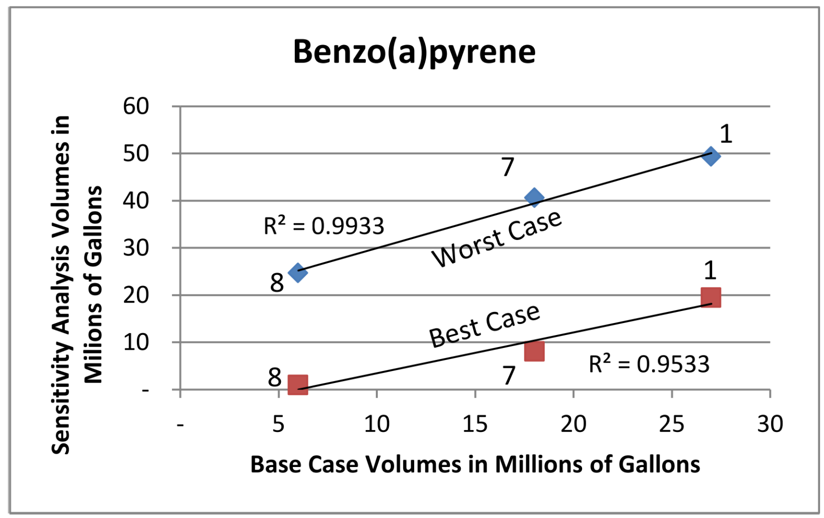
Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS | FIRM: ASPECT / DRAWN BY: JJP/SCC | FIGURE NO. A-26









CAD Path: Q:\Quendall\020027 Quendall Terminals\2013-06 FS Draft Final\020027-A-22 thru A-34.dwg A-27 || Date Saved: Oct 10, 2013 12:30pm || User: scudd

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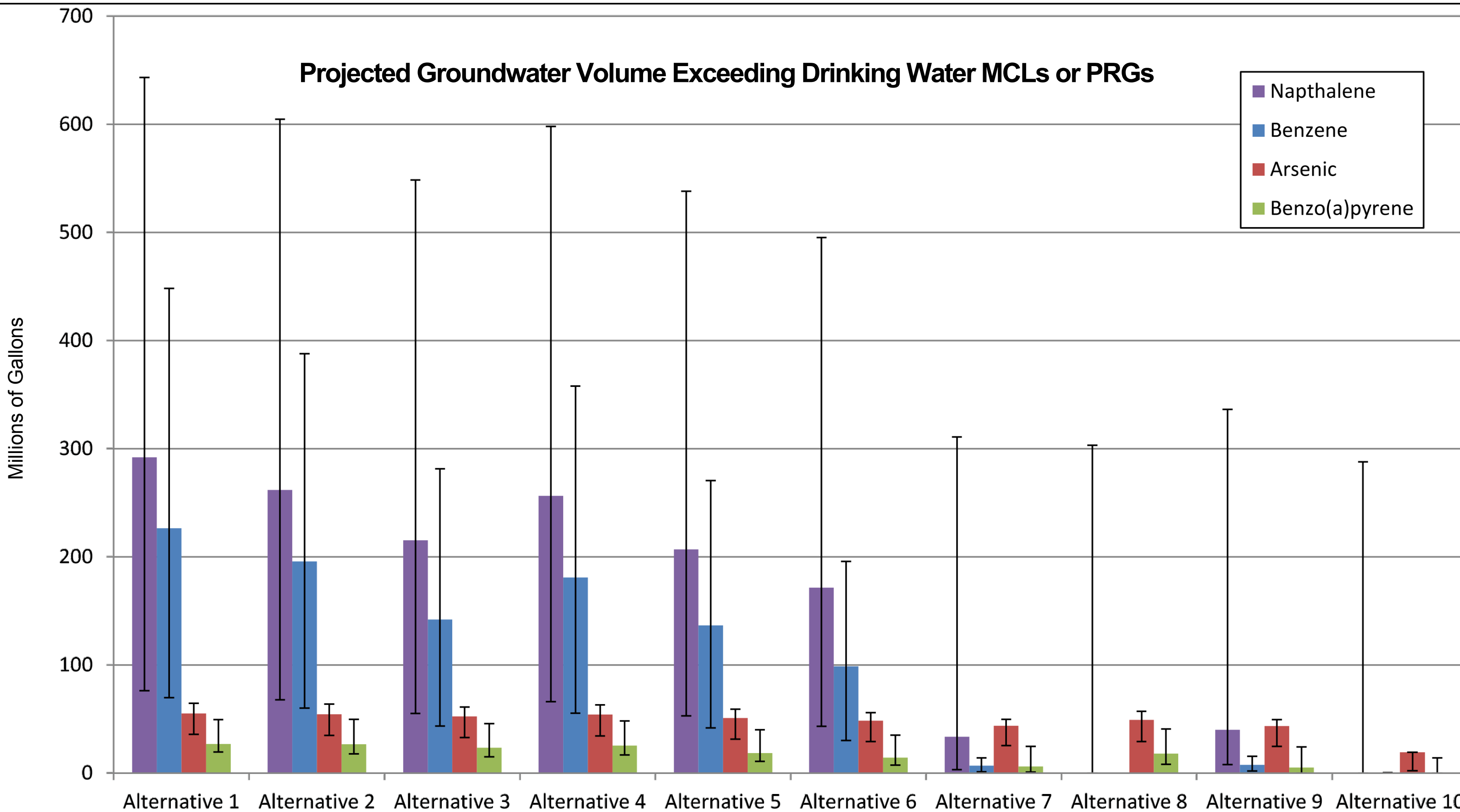
October 14, 2013

Linear Interpolation of Sensitivity Analysis Results by COC-Plume Volume

Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS | FIRM: ASPECT | DRAWN BY: JJP/SCC | FIGURE NO. **A-27**



Notes:

1. Groundwater modeling results should be used as a relative tool for comparison and not considered a prediction of actual conditions. For example, model-estimated benzo[a]pyrene volumes for all alternatives are larger than anticipated based on current site data, due to simplifying modeling assumptions (see Appendix A, Section A3.2).
2. Modeling results do not include the potential contribution of residuals resulting from removal actions (i.e., excavation or dredging). It is expected, based on a model sensitivity analysis (see Appendix A, Section A5.1.2), that residuals will result in benzo[a]pyrene exceedances after 100 years for all alternatives, including Alternative 10.
3. Benzo[a]pyrene error bar for Alternative 10 is based on volume of plume estimated under residuals sensitivity analysis (see Appendix A, Section A5.1.2).
4. Error bar represents range between best and worst cases.

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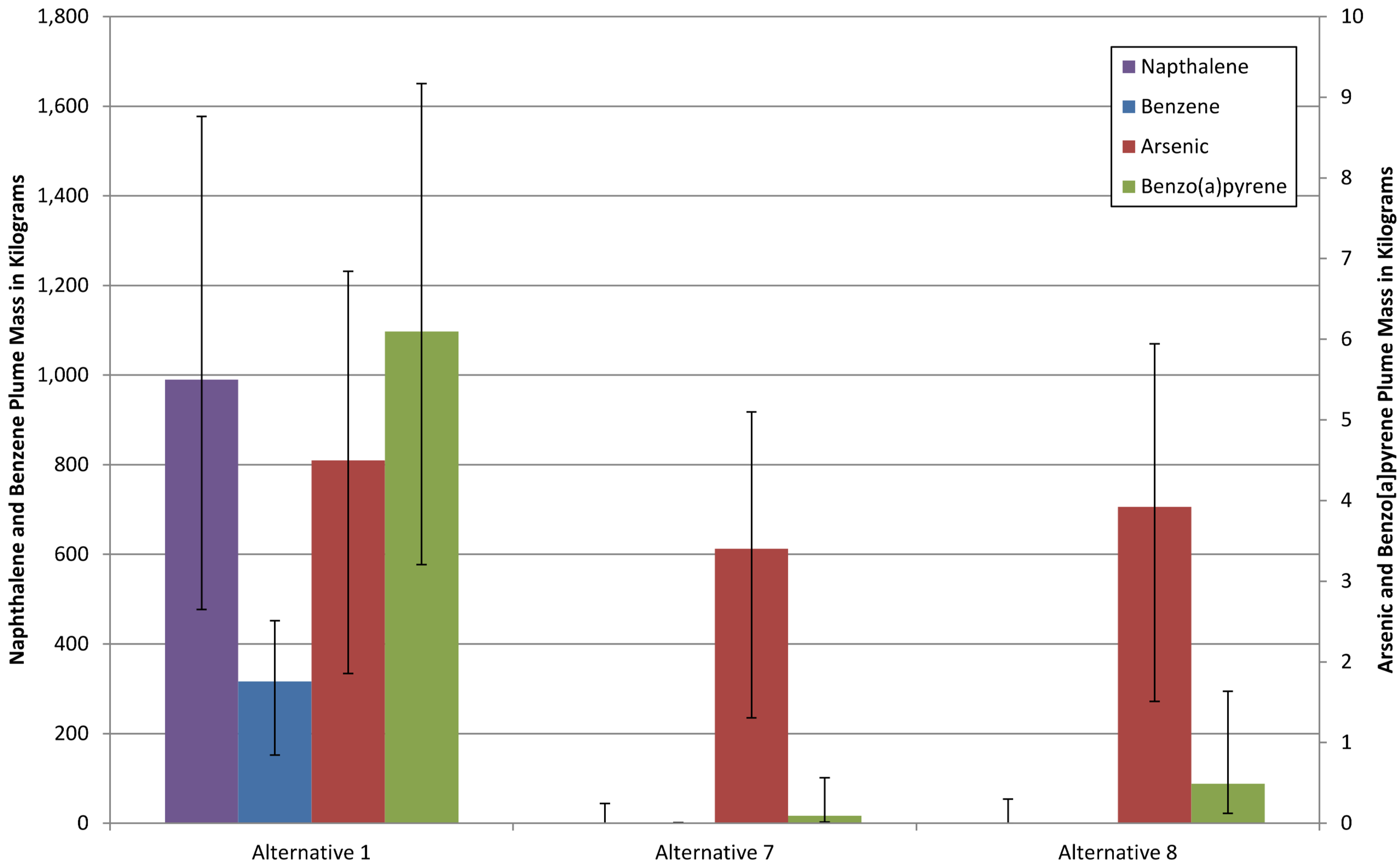
Estimated Sensitivity Analysis
Results by COC-Plume Volume
Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS | FIRM: ASPECT / DRAWN BY: JJP/SCC | FIGURE NO. **A-28**

CAD Path: Q:\Quendall\020027 Quendall Terminals\2013-06 FS Draft Final\020027-A-22 thru A-34.dwg A-28 | | Date Saved: Oct 10, 2013 12:31pm | | User: scudd



Note:
1. Error bar represents range between best and worst cases.

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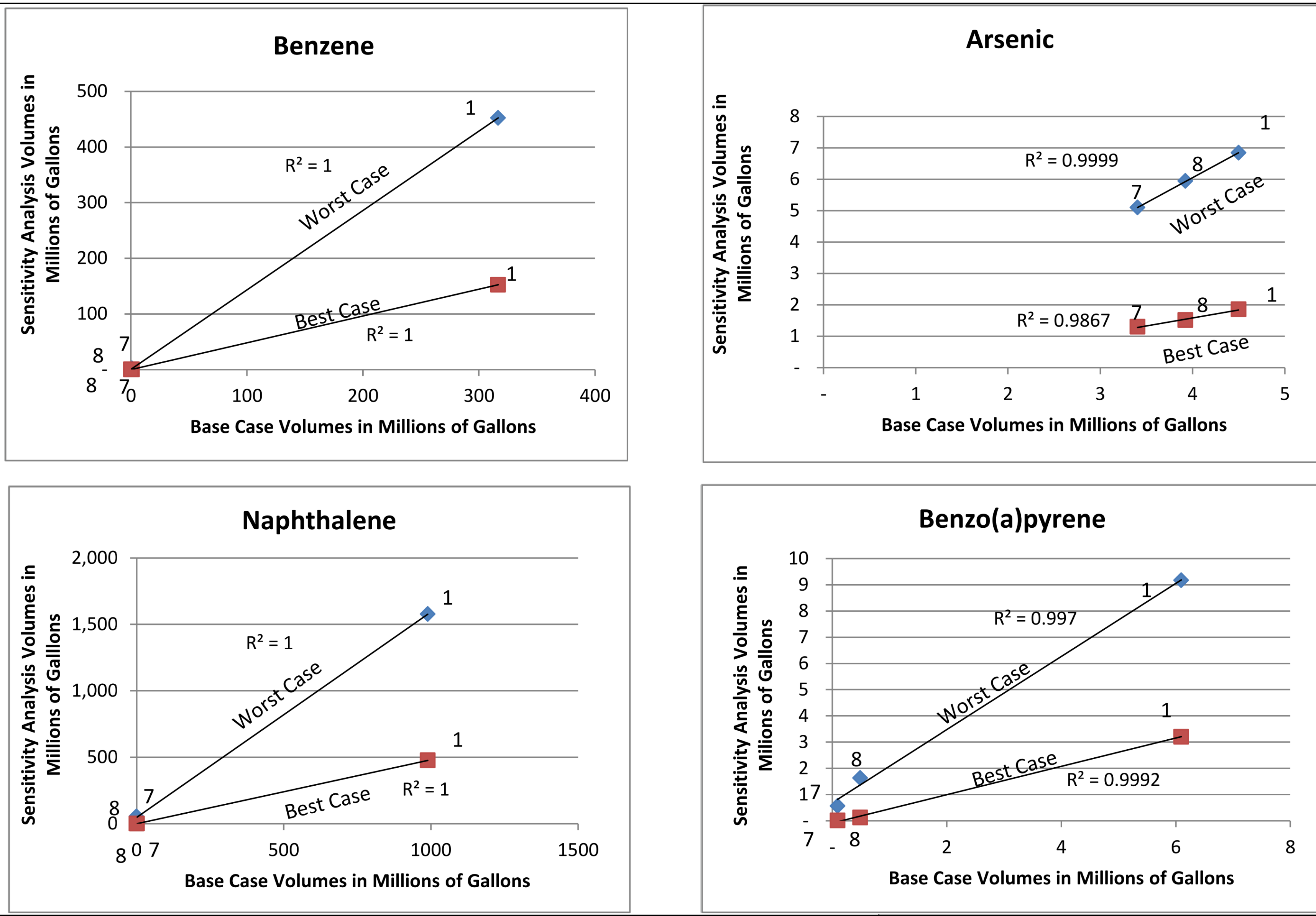
Sensitivity Analysis Results by COC Plume Mass

Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS | FIRM: ASPECT | DRAWN BY: JJP/SCC | FIGURE NO. A-29



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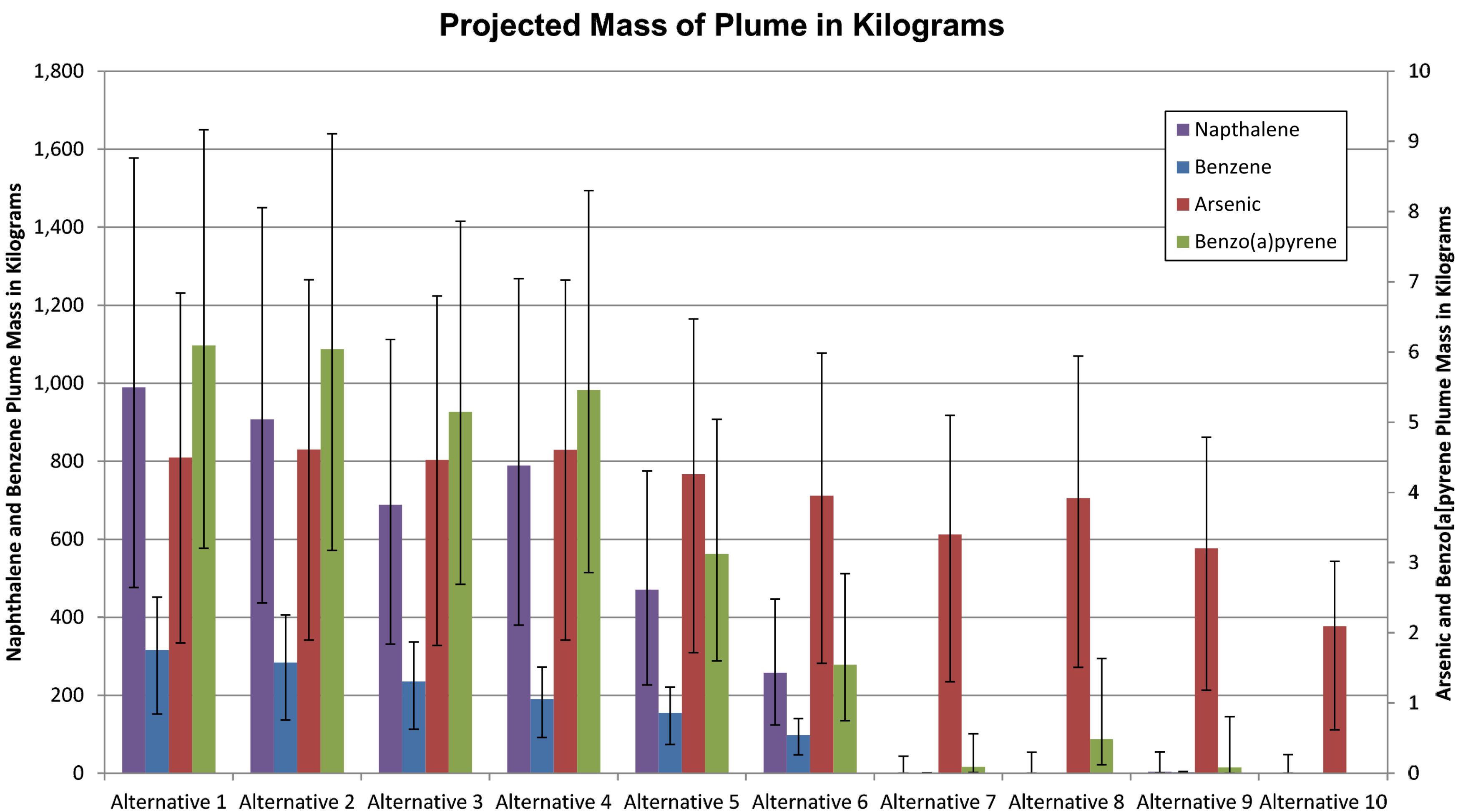
October 14, 2013

Linear Interpolation of Sensitivity Analysis Results by COC-Plume Mass

Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS | FIRM: ASPECT | DRAWN BY: JJP/SCC | FIGURE NO. A-30



Notes:

1. Groundwater modeling results should be used as a relative tool for comparison and not considered a prediction of actual conditions.
2. Modeling results do not include the potential contribution of residuals resulting from removal actions (i.e., excavation or dredging). It is expected, based on a model sensitivity analysis (see Appendix A, Section A5.1.2.2), that residuals will result in benzo[a]pyrene exceedances after 100 years for all alternatives, including Alternative 10.
3. Error bar represents range between best and worst cases.

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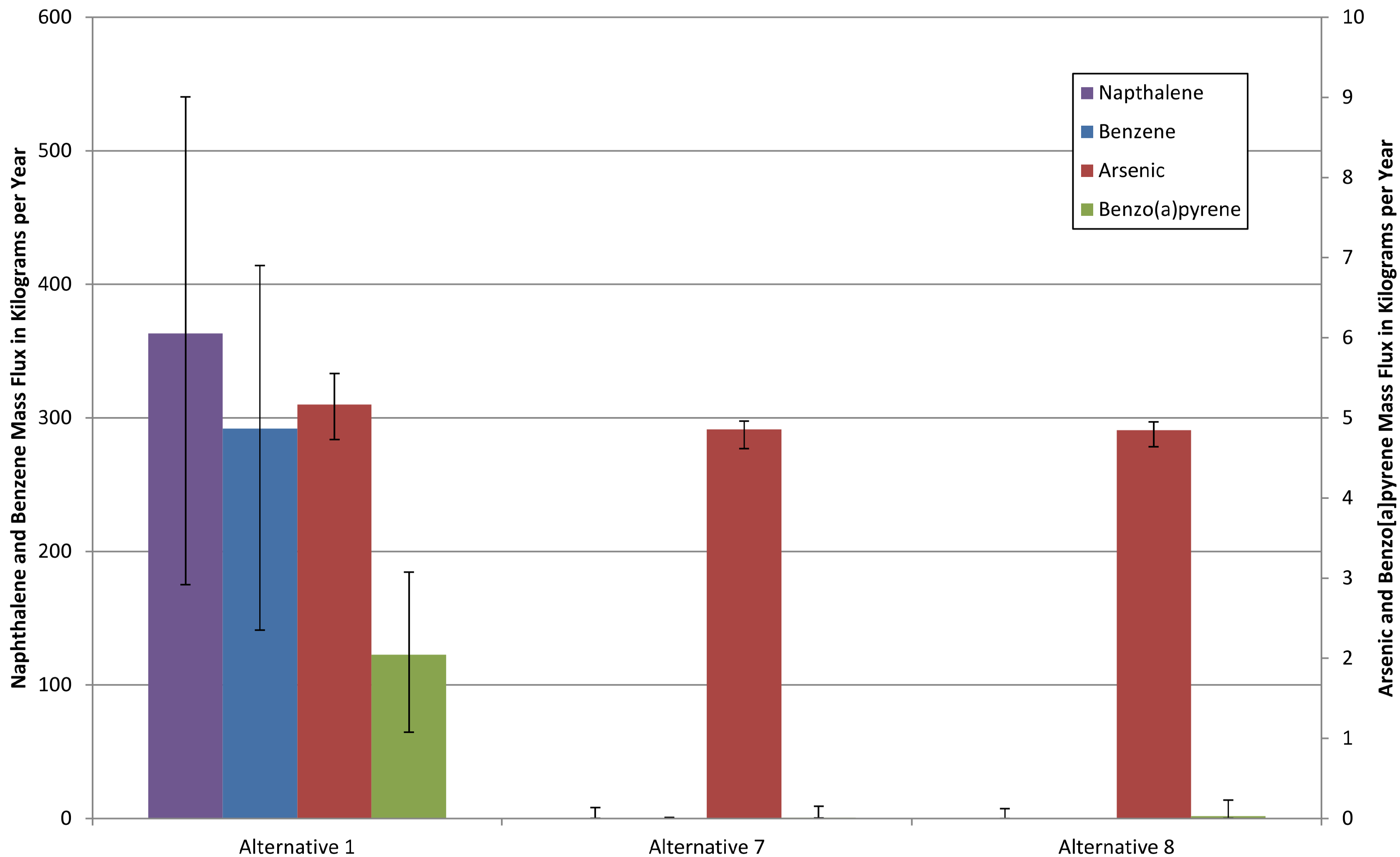
Estimated Sensitivity Analysis Results by COC-Plume Mass

Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS | FIRM: ASPECT / DRAWN BY: JJP/SCC | FIGURE NO. **A-31**

Projected Mass Flux from Model in Kilograms per Year



Note:
1. Error bar represents range between best and worst cases.

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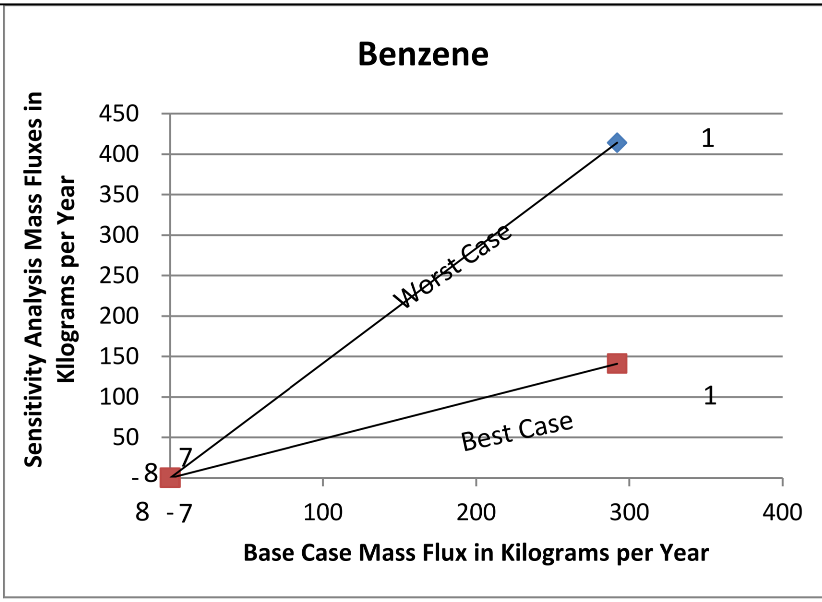
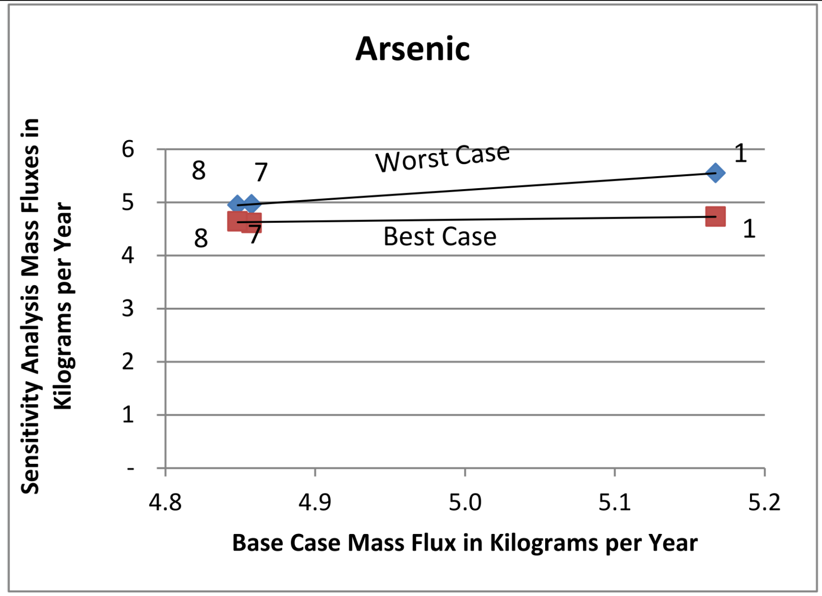
October 14, 2013

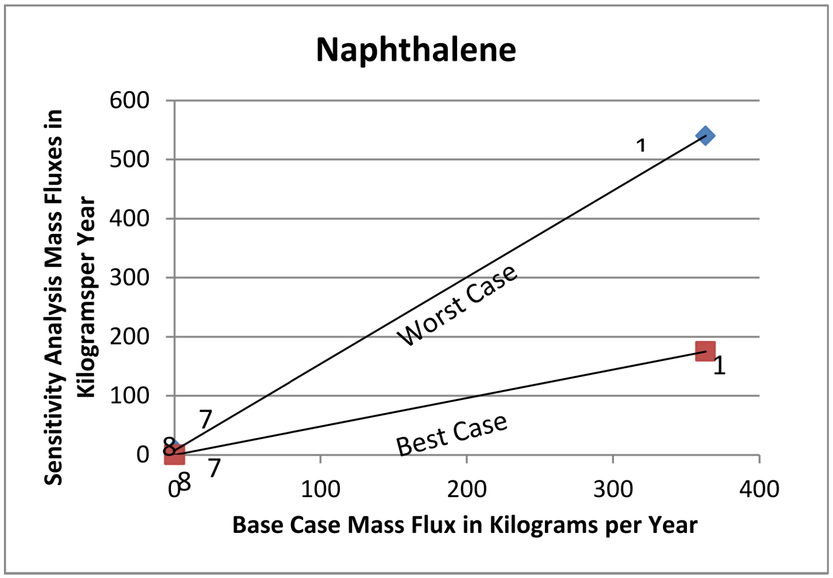
Sensitivity Analysis Results by COC Mass Flux

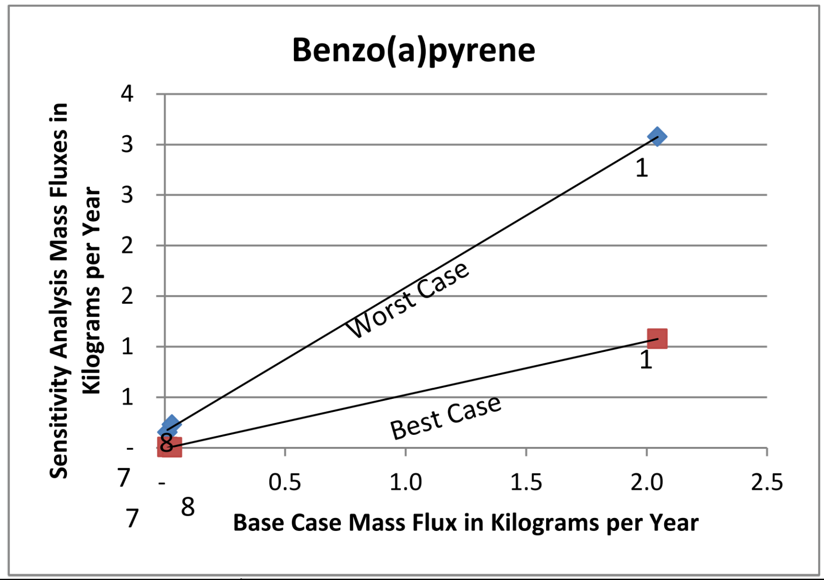
Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS | FIRM: ASPECT | DRAWN BY: JJP/SCC | FIGURE NO. A-32







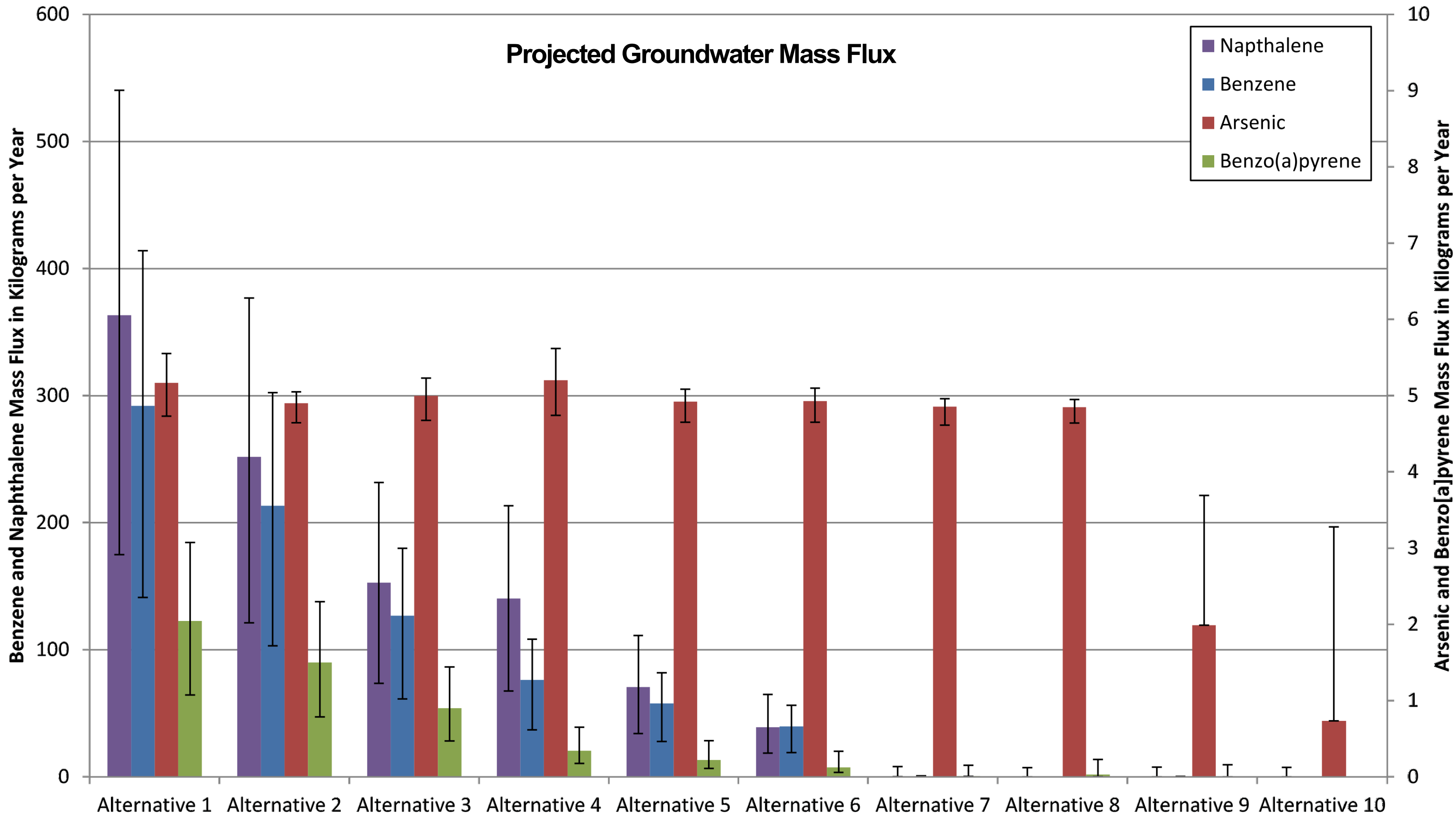


DRAFT FINAL
October 14, 2013

Linear Interpolation of Sensitivity Analysis Results by COC-Mass Flux

Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING	ARCADIS	FIRM: ASPECT DRAWN BY: JJP/SCC	FIGURE NO. A-33



Notes:

1. Groundwater modeling results should be used as a relative tool for comparison and not considered a prediction of actual conditions.
2. Modeling results do not include the potential contribution of residuals resulting from removal actions (i.e., excavation or dredging). It is expected, based on a model sensitivity analysis (see Appendix A, Section A5.1.2.2), that residuals will result in benzo[a]pyrene exceedances after 100 years for all alternatives, including Alternative 10.
3. Error bar represents range between best and worst cases.

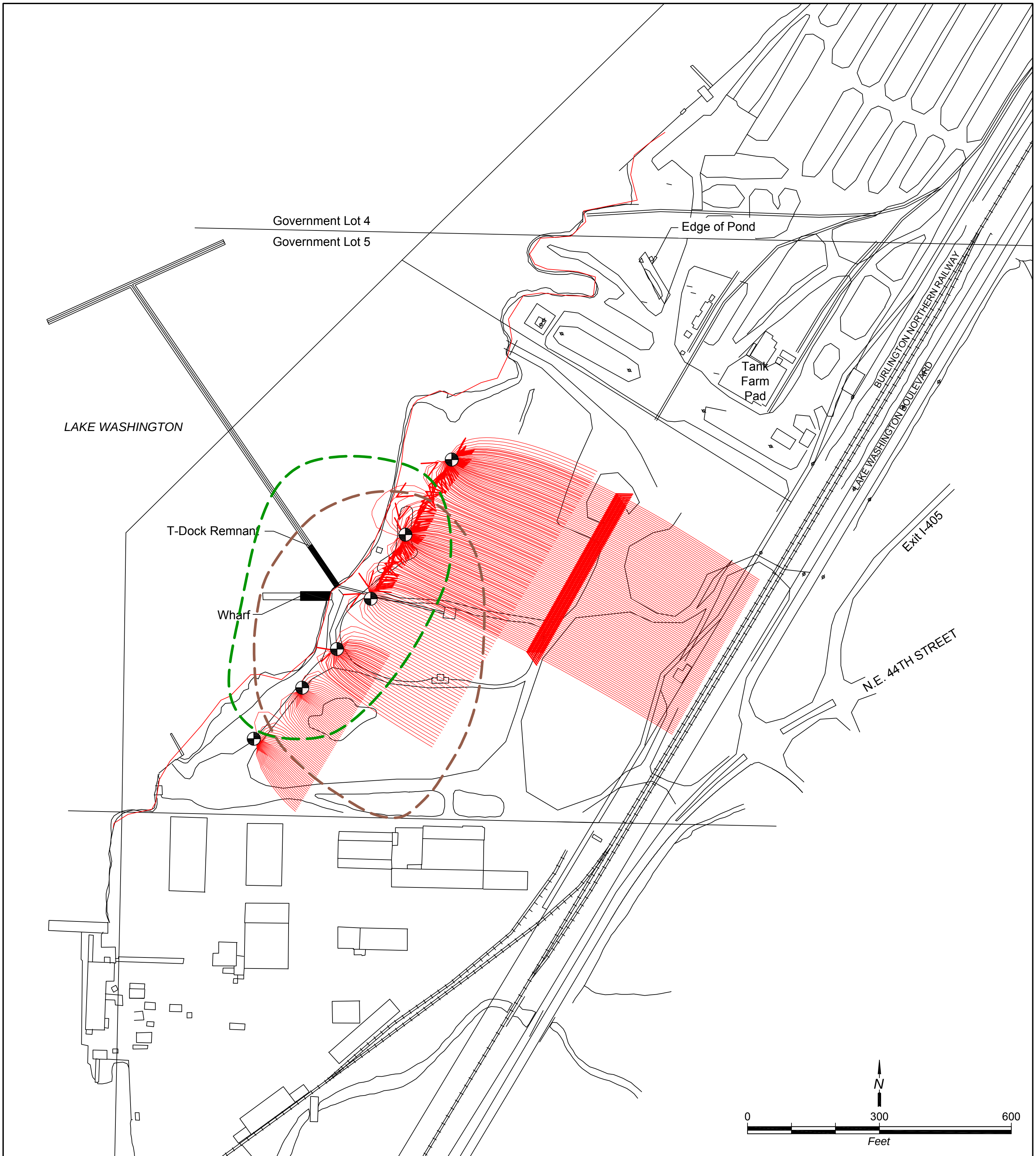
DRAFT FINAL
October 14, 2013

Estimated Sensitivity Analysis Results by COC-Mass Flux

Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS | FIRM: ASPECT | DRAWN BY: JJP/SCC | FIGURE NO. **A-34**

CAD Path: Q:\Quendall\020027 Quendall Terminals\2013-06 FS Draft Final\020027-A-22 thru A-34.dwg A-34 || Date Saved: Oct 10, 2013 12:40pm || User: scudd



LEGEND

- Pumping Well
- MODPATH Particle Trace
- Estimated extent of benzene exceeding MCL in groundwater (5 µg/L) - Deeper Alluvium
- Estimated extent of arsenic exceeding MCL in groundwater (10 µg/L) - Deeper Alluvium

DRAFT FINAL
October 14, 2013

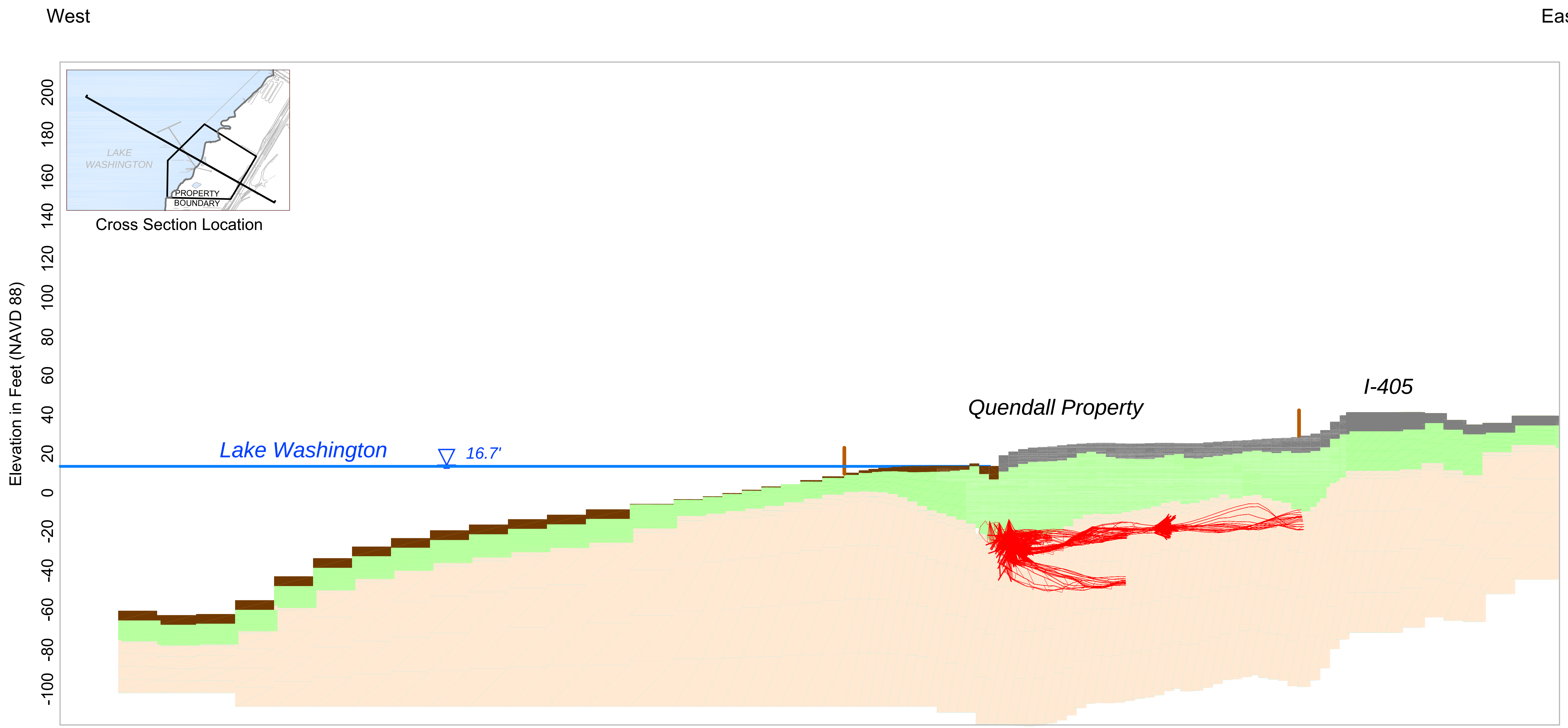
Pump and Treat Capture Analysis
Plan View

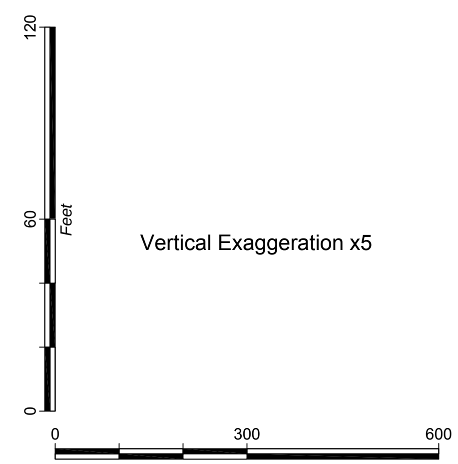
Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS

FIRM: ASPECT
DRAWN BY: SM/SCC
FIGURE NO. **A-35**

CAD Path: Q:\Quendall\020027 Quendall Terminals\2013-06 FS Draft Final_Modeling Map Figures use ACAD 2012\020027-A33.dwg Figure 1 || Coordinate System: NAD 1983 State Plane Washington North FIPS 4601 Feet || Date Saved: Sep 13, 2013 1:37pm || User: scudd





Vertical Exaggeration x5

All elevations are in feet NAVD 88.

Legend

- Fill
- Shallow Alluvium
- Deeper Alluvium
- Lake Washington Sediments
- Quendall Property Boundary
- MODPATH Particle Trace

DRAFT FINAL
October 14, 2013

Pump and Treat Capture Analysis
Cross Section View

Quendall Terminals Feasibility Study Report
Renton, Washington

Aspect CONSULTING | ARCADIS | FIRM: ASPECT | DRAWN BY: SM/SCC | FIGURE NO. A-36

APPENDIX B

Engineering Evaluations in Support of Sediment Remedial Alternatives

- B1 – cPAH Background Threshold Value and Replacement Value Calculation Memo
- B2 – Engineered Sand Cap – Chemical Isolation Layer Modeling
- B3 – Cap Armor Layer Evaluation
- B4 – Cap Geotechnical Considerations
- B5 – Sheet Pile Enclosure Calculations

APPENDIX B1

cPAH Background Threshold Value and Replacement Value Calculation Memo

ARCADIS U.S., Inc.
1100 Olive Way
Suite 800
Seattle
Washington 98101
Tel 206 325 5254
Fax 206 325 8218

MEMO

To:
Barbara Orchard
Barry Kellems

Copies:

From:
Alison Skwarski
Ryan Shatt

Date:
August 13, 2013

ARCADIS Project No.:
WA000907.0000 00003

Subject:
cPAH Background Threshold Value and Replacement Value Calculation -- Quendall Terminals Draft Final Feasibility Study

---

This memorandum summarizes the reasoning and methodology for EPA's background threshold value (BTV) of 17.5 mg/kg organic carbon (OC) for the Quendall Terminals Site (Site) and provides ARCADIS's calculations that confirm EPA's replacement value range of 15.96 to 16.3 mg/kg OC.

In the August 6, 2012 *Agency Review Draft Feasibility Study* (Draft FS) prepared for the Site by Aspect Consulting and Anchor QEA, a site-specific replacement threshold value of 27 mg/kg OC was calculated for carcinogenic polynuclear aromatic hydrocarbons (cPAHs) to determine the sediment footprint requiring remedial action. This replacement value calculation consecutively replaced the highest Site sample with a randomly generated background concentration. A t-test was used to compare the remaining Site and background datasets. A Monte Carlo simulation of the t-test was run to evaluate the probable range of the background variable to determine the mean t-test p-value and lower confidence limit and upper confidence limit on the p-value.

In EPA's April 12, 2013 comments on the Draft FS, EPA did not support the Site-specific replacement threshold of 27 mg/kg OC for cPAHs and requested that a BTV be calculated instead, based on the 95% gamma Upper Tolerance Limit (UTL) with 95% coverage of the background sample dataset. For purposes of the Remedial Investigation, background surface sediment was functionally represented by sampling data collected approximately 1 mile from the Site along the eastern Lake Washington shoreline.

Background samples were collected at similar depths and in similar depositional sediment environments to those at the Site. EPA calculated a BTV of 17.5 mg/kg OC cPAH using ProUCL 4.1, a statistical software package, and the 95% Hawkins Wixley approach.

Additionally, for comparison purposes, EPA also completed a replacement value exercise that differed from the approach used in the Draft FS. The mean background cPAH concentration of 4.62 mg/kg OC was substituted instead of a randomly generated variable. Instead of using the t-test, the Wilcoxon Rank Sum test (a non-parametric test) was used. EPA's replacement value exercise resulted in a value between 15.96 and 16.3 mg/kg OC, which was slightly lower than EPA's calculated BTV of 17.5 mg/kg OC cPAH.

EPA required that the sediment footprint requiring remedial action be based on the revised BTV of 17.5 mg/kg OC cPAH.

To confirm EPA's calculations, ARCADIS conducted the calculation using EPA's methodology and confirmed the BTV of 17.5 mg/kg OC. ARCADIS' calculation also resulted in a replacement value between 15.96 and 16.3 mg/kg OC. Attachment 1 provides the calculations used to confirm EPA's replacement value results, and Attachment 2 summarizes the calculation results.

Attachments

Attachment 1 -- cPAH Background Threshold Value and Replacement Value Calculations

Attachment 2 – cPAH Background Threshold Value and Replacement Value Results

Gamma Background Statistics for Full Data Sets

User Selected Options

From File	WorkSheet_a.wst
Full Precision	OFF
Confidence Coefficient	95%
Coverage	95%
Number of Bootstrap Operations	2000

Background ug/kg OC

Raw Statistics

Number of Valid Observations	10
Number of Distinct Observations	10
Minimum	1709
Maximum	13022
Second Largest	5808
Mean	4620
Geometric Mean	3912
First Quartile	2631
Median	3870
Third Quartile	5144
SD	3250

Gamma Distribution Test

k hat (MLE)	3.162
k star (bias corrected MLE)	2.28
Theta Hat (MLE)	1461
Theta star (bias corrected MLE)	2026
nu hat (MLE)	63.25
nu star (based upon bias corrected estimates)	45.61
MLE Mean (based upon bias corrected estimates)	4620
MLE Sd (based upon bias corrected estimates)	3059
95% Percentile of Chisquare (2k)	10.38
A-D Test Statistic	0.404
5% A-D Critical Value	0.732
K-S Test Statistic	0.172
5% K-S Critical Value	0.268

Data appear Gamma Distributed at 5% Significance Level

Background Statistics Assuming Gamma Distribution

90% Percentile	8715
95% Percentile	10519
99% Percentile	14489
95% Wilson Hilferty (WH) Approx. Gamma UPL	11160
95% Hawkins Wixley (HW) Approx. Gamma UPL	11286
Tolerance Factor K	2.911
95% Wilson Hilferty (WH) Approx. Gamma UTL with 95% Coverage	16764
95% Hawkins Wixley (HW) Approx. Gamma UTL with 95% Coverage	17494

Nonparametric Background Statistics

95% Chebyshev UPL	19477
95% BCA Bootstrap UTL with 95% Coverage	13022
95% Bootstrap (%) UTL with 95% Coverage	13022

Attachment 2 of Appendix B1 - cPAH Background Threshold Value and Replacement Value Results

Quendall Terminals
Renton, Washington

Iteration	Number of Background Replacements	Sample ID	Concentration (mg/kg-OC)	Untransformed Data Wilcoxon p-value	Log-Transformed Data Wilcoxon t-test p-value	Mean (mg/kg-OC)	Standard Deviation
1	0	TD-15-BS	2,370	3.11E-05	2.71E-05	186	532
2	1	TD-08-BS	1,771	5.32E-05	4.66E-05	104	326
3	2	NS-12-BS	278	8.95E-05	7.87E-05	43.4	60.9
4	3	NS-11-BS	176	1.48E-04	1.31E-04	34.0	41.4
5	4	TD-14-BS	130	2.42E-04	2.14E-04	28.1	31.4
6	5	SS-05-BS	98.3	3.88E-04	3.45E-04	23.8	24.8
7	6	SS-06-BS	81.8	6.14E-04	5.48E-04	20.5	20.4
8	7	SS-04-BS	66.2	9.55E-04	8.57E-04	17.9	16.9
9	8	TD-09-BS	61.8	0.00146	0.00132	15.8	14.3
10	9	TD-11-BS	38.2	0.00221	0.002	13.8	11.3
11	10	NS-07-BS	38.1	0.00329	0.00298	12.6	10.4
12	11	NS-16-BS	38.1	0.00481	0.00438	11.5	9.30
13	12	TD-SO-03-SS-09093	30.9	0.00694	0.00634	10.3	7.84
14	13	TD-13-BS	27.8	0.00985	0.00904	9.41	6.83
15	14	TD-SO-02-SS-09093	22.9	0.0138	0.0127	8.61	5.89
16	15	SS-03-BS	20.8	0.019	0.0176	7.98	5.25
17	16	TD-12-BS	18.7	0.0258	0.024	7.42	4.67
18	17	TD-CT-02-SS-09093	16.5	0.0346	0.0322	6.93	4.16
19	18	TD-SO-01-SS-09093	16.3	***0.0457***	***0.0426***	6.53	3.74
20	19	TD-CT-01-SS-09093	16.0	***0.0594***	***0.0557***	6.12	3.25
21	20	TD-10-BS	13.1	0.0762	0.0717	5.73	2.65
22	21	TD-SO-04-SS-09093	11.2	0.0964	0.091	5.44	2.25
23	22	TD-07-BS	10.7	0.114	0.108	5.21	1.97
24	23	TD-SO-05-SS-09093	10.3	0.134	0.127	5.01	1.66
25	24	NS-03-BS	10.0	0.155	0.148	4.81	1.31
26	25	TD-SO-09-SS-09093	6.94	0.18	0.171	4.62	0.84
27	26	TD-SO-07-SS-09093	5.44	0.206	0.197	4.54	0.71
28	27	TD-SO-08-SS-09093	5.15	0.225	0.215	4.51	0.69
29	28	TD-SO-06-SS-09093	0.98	0.235	0.235	4.49	0.68

Background Sample	Concentration cPAH (mg/kg-OC)	Concentration Log cPAH
BG-03-BS	13.0	1.11
BG-13-BS	5.81	0.76
BG-04-BS	5.16	0.71
BG-06-BS	5.09	0.71
BG-19-BS	3.89	0.59
BG-09-BS	3.85	0.59
BG-12-BS	2.83	0.45
BG-17-BS	2.57	0.41
BG-15-BS	2.27	0.36
BG-02-BS	1.71	0.23
Average Background	4.62	0.59

Notes:

1. The first iteration used all data; no samples were replaced. The second iteration replaced the highest sample concentration with average background value. The third iteration replaced the two (2) highest sample concentrations with the average background value and so on for all the data.
2. Each iteration of sample concentrations was statistically compared against the background concentrations in ProUCL 4.1 using the Wilcoxon-Mann-Whitney test (95% confidence and a null hypothesis of area of concern [AOC] ≤ Background).
3. Log transformed p-values used the same dataset iterations but the concentrations used are the log values.
4. The highlighted cells indicate the results that bracket a Wilcoxon Rank Sum Test p-value of 0.05.

cPAH = carcinogenic polynuclear aromatic hydrocarbon
mg/kg-OC = milligram(s) per kilogram organic carbon

APPENDIX B2

Engineered Sand Cap – Chemical Isolation Layer Modeling

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List of Figures

Attachments

B2-1 Introduction

In support of the Quendall Terminals Site (Site) feasibility study (FS), one-dimensional chemical mass transport modeling was performed to develop a conceptual-level chemical isolation layer design for an engineered sand cap included in remedial alternatives evaluated in the FS. The engineered sand cap modeling was performed using analytical model tools and assumptions following guidance for designing sediment caps developed by both the U.S. Army Corps of Engineers (USACE; Palermo et al. 1998) and the U.S. Environmental Protection Agency (EPA 2005).

The chemical isolation layer modeling was initially applied to measured sediment porewater cation profiles at the Site, using validated Site characterization data presented in the Remedial Investigation (RI) Report (Anchor QEA and Aspect 2012). The model in this application provides a useful analytical framework to help differentiate the combined effects of a range of physical processes (e.g., advection and dispersion) from chemical and biological degradation processes for Site chemicals of concern (COCs).

Following simulation with best-estimate values for the range of physical parameters and calibration of degradation parameters to existing Site conditions, the analytical model was subsequently applied to simulate the effectiveness of an engineered sand cap in reducing long-term flux of COCs into surface sediments and achieving the surface water/porewater and surface sediment preliminary remediation goals (PRGs) summarized in Tables 4-6 through 4-7 in the main body of this FS. To provide a conservative assessment of long-term cap effectiveness consistent with the USACE and EPA guidance, long-term cap protectiveness was defined in this FS evaluation at steady-state conditions (i.e., infinite timeframe), also conservatively assuming that the current soil and groundwater “source” concentrations to Site sediments do not diminish over time. Thus, cap designs developed using these and other conservative assumptions as described in this appendix are projected to achieve PRGs under steady-state conditions. The cap isolation layer modeling evaluation was performed for the shallow nearshore sediment area at the Site as depicted on Figure 6-1 of the main FS report.

B2-2 Methodology

B2-2.1 Model Framework

The one-dimensional analytical steady-state model of chemical transport within sediment caps developed by Dr. Danny Reible from the University of Texas (as described in Lampert and Reible 2009, and Reible 2012) was used for this evaluation (hereafter referred to as the UT model). Although this model was originally developed to simulate sediment caps, it can also be applied to represent uncapped conditions. Predictions calculated using the steady-state model provide a useful means of assessing long-term COC profiles within a subaqueous sediment/cap system, although the time to reach the steady-state concentrations predicted by the model will vary, depending on the chemical characteristics of the COCs, sediment geochemical conditions, and subsurface hydrogeology.

As shown on Figure B2-1, the UT model consists of two layers: a chemical isolation layer and a bioturbation zone. The UT model conservatively assumes that soil and groundwater COC concentrations underlying the sediments remain constant over time (i.e., infinite source); therefore, detailed simulation of transport within the underlying soils and groundwater is not necessary in this application. COC concentrations in surface water overlying the sediments are treated as a boundary condition in the UT model (it is typically assumed to be zero, which is usually appropriate in the case of sorptive contaminants, but that assumption was refined for this FS analysis in certain cases, as discussed below). The groundwater transport mechanisms of advection, diffusion, dispersion, partitioning between the aqueous and sorbed (sediment or cap material) phases, and first-order reaction (to represent degradation processes) are all incorporated into the model. In addition, the model incorporates mass transfer processes at the sediment-water interface, including biological mixing and exchange through the benthic boundary layer with the overlying water column.

The UT model calculates steady-state porewater and sorbed phase COC concentrations vertically throughout the cap (or existing sediment when the model is used to represent current uncapped conditions), including the surficial (bioturbation) zone. As dissolved COCs move upward through the cap through advection and diffusion, they can undergo degradation while at the same time partitioning onto the solid phase. Bioturbation mixes the surface layer, further reducing surface concentrations. The UT model calculates COC concentrations in the bioturbation zone as a balance between the flux from the underlying chemical isolation layer, the flux associated with bioturbation processes, and the flux leaving the benthic boundary layer and entering the overlying water column.

Details on the UT model structure, its underlying theory, and the governing equations, including the analytical steady-state solution, are provided in Lampert and Reible (2009). Additional details on other similar one-dimensional models of sediment caps are provided in Go et al. (2009) and the USACE/EPA capping technical guidance document (refer to Appendix B of Palermo et al. 1998).

B2-2.2 Approach

The UT model was used to predict steady-state COC concentrations at the surface of the cap for assessing preliminary engineered sand cap design options and long-term cap effectiveness in nearshore areas. The general approach used to perform the modeling is outlined below:

- Initial modeling was performed looking at current Site conditions to assess the appropriateness of the best-estimate literature values for the parameters that describe the various physical processes occurring at the Site (based on observed porewater cation concentration profiles). The model was then calibrated to develop estimates of the parameters that describe the various chemical and biological processes occurring at the Site (based on observed porewater COC concentration profiles. The details of the approach and results from this modeling are provided in Section B2-3.3.

- To evaluate the long-term effectiveness of the engineered sand cap, the calibrated model was configured to represent preliminary cap design and projected changes in groundwater flux in the sediment areas that would occur following construction of the remedy. Steady-state sediment porewater COC concentrations modeled within the upper zone of the cap were then compared to PRGs for the two most mobile Site COCs—naphthalene and benzene. The details of the approach and results from this component of the chemical isolation layer modeling are provided in Section B2-4.3.

B2-3 Initial Modeling

B2-3.1 Approach

The UT model was used in this FS evaluation to help quantify the combined effects of a range of physical processes that occur in the Site sediments. This was accomplished by configuring the model to simulate measured concentration profiles of cations, which behave largely as non-reactive tracers at the Site.

The model coefficients were specified based on Site-specific data, where available, or literature values for similar conditions. Since many of the parameters were not readily available for the Site-specific conditions, the best available literature value or typical modeling value was used but there remains a degree of uncertainty. Some of these parameters are fairly well established and exhibited little variability or result in minimal variability of model output (e.g., diffusion coefficients). Other parameters related to particle dynamics may be significant to organic compounds which sorb to sediments, but will not appreciably influence dissolved cations.

Once the parameters were specified, the model simulations were run for cations. Model output was compared to the cation porewater data collected from the nearshore area of the Site (Anchor QEA and Aspect 2012) to see if the model predictions matched the measured vertical profiles of the porewater cation data. The cation simulation has the advantage of being able to exclude degradation reactions (and for the most part partitioning) which impact the COCs, allowing the cation model simulations to focus on applicability of the physical parameters. Given the number of unknown or uncertain parameters, the input parameters in the cation model were not calibrated, but rather the model was used to determine if the best estimates for the unmeasured parameters yield a reasonable match. If so those values would then be carried on to simulations of the COCs.

Based on the acceptability of the cation model prediction, the model was then used to simulate porewater benzene and naphthalene concentrations. Chemical-specific coefficients (diffusivity in water and organic carbon partition coefficients) were changed when the physical/ chemical model was converted from simulating cations to simulating benzene and naphthalene. The UT model was then calibrated to fit the measured porewater benzene and naphthalene profile data (Anchor QEA and Aspect 2012) by increasing degradation rates for these COCs.

B2-3.2 Model Inputs

Specification of input parameters for the current conditions model was based on Site-specific data, such as groundwater flow velocities and porewater benzene and naphthalene concentration profiles, along with information from the literature and experience with modeling other similar sites. Physical parameter model inputs were checked by their use in the cation model and results compared with the Site-specific cation data. Similarly, degradation rates for benzene and naphthalene were determined through calibration of the UT model against measured existing conditions. Details on the development of the various model input parameters are provided in the following sections.

B2-3.2.1 Input Parameters Based on Site Data and Literature

B2-3.2.1.1 Thickness of Model Domain

The sediment thickness evaluated in the current conditions modeling was 40 centimeters (cm; 1.3 feet), which represents the average depth of the greatest COC concentrations observed in the samples collected during the RI in the nearshore area from which cation, benzene, and naphthalene porewater data were collected. The top 8 cm of the modeled thickness was represented as the bioturbation zone. This thickness is typical of the median depth in estuarine systems (Thomas et al. 1995)

B2-3.2.1.2 Initial Porewater Concentrations

The UT model works under the assumption that the overlying surface water constituent concentrations are negligible. While this assumption is appropriate for benzene and naphthalene (given their volatility and low surface water concentrations), the cation data exhibit non-zero concentrations in Site surface water (Table B2-1). To allow for simulation of the porewater cation concentration profiles in the sediment, the concentrations measured within the porewater were corrected to be relative to the surface water concentration (to satisfy the model-assumed zero surface water concentration) and normalized to the concentration at depth using the following equation:

$$C_{N(i)} = \frac{C_{PW(i)} - C_{SW}}{C_{PW(\max)} - C_{SW}}$$

Where:

i	=	index for depth interval
C_N	=	the normalized concentration in mg/L
C_{PW}	=	the concentration in porewater in mg/L
C_{SW}	=	the concentration in the surface water in mg/L
$C_{PW(max)}$	=	the cation concentration collected from the depth of maximum concentration (40 cm average) in mg/L

Table B2-1 – Cation Porewater Concentrations

Depth in cm	Potassium in mg/L	Sodium in mg/L	Calcium in mg/L	Magnesium in mg/L	Average Cation Concentration in mg/L
Original Data					
Surface water	0.9+/-0.0	4.2+/-0.06	9.0+/-0.09	3.4+/-0.04	
0-10	2.2+/-0.41	8.2+/-1.4	21.6+/-3.6	8.2+/-2.2	
40	3.1+/-0.32	15.7+/-1.3	26.4+/-3.5	11.3+/-1.8	
Normalized Data (unitless)					
0-10	0.59+/-0.118	0.35+/-0.12	0.72+/-0.20	0.61+/-0.27	0.57+/-0.19
40	1.00+/-0.14	1.00+/-0.12	1.00+/-0.20	1.00+/-0.23	1.00+/-0.13

Note:
Porewater concentrations are based on nearshore data; average values +/- standard error are shown.

The measured Site porewater cation concentration profiles, including the normalized concentrations used in the UT model, are summarized in Table B2-1. The model input (boundary condition) was set to the normalized concentration at the 40 cm depth, which was equal to 1. The normalized concentrations for the 0 to 10 cm depth interval was averaged across the four individual cations (Table B2-1) were used to calibrate the Site-specific coefficients.

The measured benzene and naphthalene porewater concentrations in each sampled depth interval are summarized in Table B2-2. These data are summarized as the average measured (i.e., not normalized) concentrations at three sampled depths. Table B2-2 was generated using benzene and naphthalene data from near shore surface grab samples (e.g. NS-04-SS) for depth of 0-10 cm and data from nearshore vibracore samples for depths 40 cm and 125 cm. Only the vibracore sample locations with available collocated surface grab sample locations were used in generating Table B2-2. The greatest concentrations are generally observed at 40 cm depth. The average concentrations from the 40 cm sampling depth were used to specify the initial porewater concentration used for the current conditions simulations of these COCs.

Table B2-2 – Benzene and Naphthalene Porewater Concentrations

Depth in cm	Benzene in µg/L	Naphthalene in µg/L
0-10	0.46+/-0.22	1.19+/-0.49
40	200+/-199.9	106.6+/-105.3
125	134.4+/-123.8	3.4+/-1.4

Notes:
Porewater concentrations are based on nearshore data; average values +/- standard error are shown.
For non detects, half of the reporting limit values was used for averaging.
Samples from the depth range of 8 – 12 inches were used for 40 cm depth.
Samples from the depth ranges of 20 – 24 inches and 36 -40 inches were used for 125 cm depth.

B2-3.2.1.3 Groundwater Seepage Velocity

The numerical groundwater flow model developed for the Site was used to calculate groundwater seepage velocities through Site sediments under existing conditions (Table

A-8, in Appendix A). The flow model calculated an average groundwater seepage velocity of 176 centimeters per year (cm/yr) in nearshore areas of the Site, which was used as a base case. These flow calculations were corroborated with Site-specific measurements of lake bed seepage (Table 3.1-3; Anchor QEA and Aspect 2012). Due to the suspicion that an overestimation of groundwater seepage flux may in turn be resulting in an overestimation of COC degradations rates, the current conditions modeling was also conducted using groundwater seepage velocities ranging from 66 to 176 cm/yr. The paired seepage and resultant degradation rates were evaluated as sensitivity cases.

B2-3.2.1.4 Physical Parameters

The selection of various physical parameter values such as boundary layer mass transfer coefficient (K_{bl}) , dispersivity (alpha), particle biodiffusion coefficient ($D_{bio}{}^{p}$) and porewater biodiffusion coefficient ($D_{bio}{}^{pw}$) in the model is outlined below.

The mechanical dispersion of a chemical through the cap is modeled as a Fickian Diffusion-like process. The dispersion coefficient is related to the product of the groundwater velocity through the cap and a length scale related to the size of the domain considered (Neuman 1990) A value of 4 cm was selected for alpha, the dispersivity coefficient, based on the 40 cm sampling depth, and an estimated 10 percent factor consistent with values from Neuman (1990) for a domain of approximately 1 meter.

The boundary layer mass transfer coefficient dictates the transport at the cap-water interface. Boudreau and Jorgensen (2001), Thidodeaux (1996) and Thibodeaux et al. (2001) present empirical values to estimate this parameter. A common value of 1 centimeter per hour (cm/hr) is frequently used for capping simulations of highly hydrophobic compounds. However, the literature indicates that the mass transfer coefficient is a function of a chemical's hydrophobicity, exhibiting a positive relationship with the partition coefficient (Thibodeaux et al. 2001); therefore, smaller values would be expected for benzene and naphthalene. The input value of 0.33 cm/hr used for the model was selected as a value typical of a compound with partitioning coefficient on the order of 10^3 (Thibodaux et al. 2001).

The process of bioturbation serves to increase the effective diffusion/dispersion coefficient for mass transport. Thomas et al. (1995) and Thibodeaux (1996) provided an extensive review of measured particle biodiffusion coefficient ($D_{bio}{}^{p}$) and porewater biodiffusion coefficient ($D_{bio}{}^{pw}$) at different locations in the United States. The value of 9 cm^2/yr used in the model for $D_{bio}{}^{pw}$ is the median value observed in estuarine conditions (Thomas et al. 1995) and consistent with the range of value for marine conditions presented in Thibedeaux (1996). There is less guidance regarding the value of $D_{bio}{}^{p}$ which was selected to be 100 times $D_{bio}{}^{pw}$ as suggested by Lampert and Reible (2009, resulting in a value of 900 cm^2/yr. Again this value is consistent with the range of values for marine conditions presented in Thibodeaux (1996).

B2-3.2.2 Partitioning Coefficients

Partitioning of chemicals between the dissolved and sorbed phases is described in the UT model by the chemical-specific equilibrium partition coefficient (K_d) based on the customary $K_d = f_{OC}*K_{OC}$ approach (e.g., Karickhoff 1984), where K_{OC} is the compound's organic carbon partition coefficient and f_{OC} is the organic carbon content of the solid phase material (i.e., sediment). The log K_{OC} value used in the model for simulation of cations was set to a nominally low value because these species, as tracers, do not readily associate with the particulate phase. In the model, the octanol-water partition coefficient (log K_{OW}) is used to estimate log K_{OC} (log K_{OC} = 0.903*log K_{OW} + 0.094). The partition coefficients (log K_{OW}) used in the current conditions simulations of benzene and naphthalene were 2.13 and 3.29, respectively.

B2-3.2.3 COC Calibration

Benzene and the naphthalene degradation half-lives in surface and near-surface sediments at the Site under existing conditions were estimated by increasing the degradation rate from the base value of zero until the model-predicted concentrations matched the measured Site COC concentration profiles.

B2-3.2.4 Model Input Summary

A full listing of the model input parameters used for simulation of both cations and COCs (benzene and naphthalene) is presented in Table B2-3. This table is divided into sections containing input parameters that are general to each chemical modeled and those that are chemical-specific.

Table B2-3 – UT Model Input Parameters Used in Cation, Benzene, and Naphthalene Calibrations **Sheet 1 of 2**

Model Input Parameters	Value	Notes
Porosity, e	0.4	Typical value for surface and subsurface sediments.
Bioturbation Layer Thickness, h_{bio} in cm	8	Typical value used in cap modeling for marine environments.
Cap Material Type	C	Based on observations of sediment type, the sediment was specified as consolidated (silt or clay) material (C), which causes the model to calculate the effective molecular diffusion coefficient as a function of porosity based on the formulation of Boudreau (1997).
Depositional Velocity, Vdep in cm/yr	0.5	Average depositional velocity based on radionuclide-dated cores (Table 4-3 in Anchor Environmental and Aspect 2004).
Darcy Velocity, Vdar (positive is upwelling) in cm/yr	Base value: 176 Sensitivity Range: 66 - 176	Darcy velocities representative of nearshore conditions. Values are based on results of the calibrated groundwater model combined with local variations in material type (Table A-8, in Appendix A).
Particle Density, ρ_P in g/cm^3	2.5	Typical value for sediment particles (e.g., Domenico and Schwartz 1990).
Biological Active Zone fraction organic carbon, $(f_{oc})_{bio}$	8%	Average value from top 8 cm of the sediments at the Site.
Fraction organic carbon, $(f_{oc})_{eff}$	4%	Average values from sediment depths between 10 and 100 cm at the Site.
Dispersivity, α in cm	Base value: 4	Values were determined through calibration to cation data (10% of modeled depth).
Boundary Layer Mass Transfer Coefficient, Kbl in cm/hr	Base value:0.33	Values were determined through calibration to cation data.

Table B2-3 – UT Model Input Parameters Used in Cation, Benzene, and Naphthalene Calibrations **Sheet 2 of 2**

Model Input Parameters	Value			Notes
Porewater Biodiffusion Coefficient, D_{biopw} in cm^2/yr	900			Parameter represents bioturbation rate applied to dissolved phase. Typical value used for capping design of marine environments based on Thibodeaux (1996).
Particle Biodiffusion Coefficient, D_{biop} in cm^2/yr	9			Parameter represents bioturbation rate applied to particulate phase. Typical value used for capping design as 1% of Porewater Biodiffusion Coefficient.
Modeled depth in cm	40			Based on average depth of greatest porewater concentrations observed.
Chemical-Specific Parameters	**Cations**	**Naphthalene**	**Benzene**	**Notes**
Contaminant Initial Porewater Concentration, C_0 in µg/L	1	106	200	Cation model results are simulated in normalized space relative to the surface water concentration; therefore, the initial C_0 value was set to 1 (see Section B2-3.2). Porewater values for naphthalene and benzene are nearshore averages reported for deeper subsurface sediments (40 cm).
Organic Carbon Partition Coefficient, log K_{OW}	-1	3.29	2.13	Typical values from literature.
Colloidal Organic Carbon Partition Coefficient, log K_{DOC} in log L/kg	NA	NA	NA	Partitioning to dissolved organic carbon (DOC) was not considered as it is generally not important for cations or the relatively less sorptive contaminants (i.e., naphthalene and benzene) evaluated in the model.
Colloidal Organic Carbon Concentration, r_{DOC} in mg/L	NA	NA	NA	
Water Diffusivity, D_w in cm^2/sec	2.5E-05	4.7E-06	6.0E-06	Cation values estimated using correlation identified from Schwarzenbach et al. (1993), relating diffusivity to a compound's molecular weight. Benzene and naphthalene values from Lyman et al. (1990).
Undifferentiated chemical and biological degradation half life, λ_1 in days	0	Base value: 7 Sensitivity range: 7 - 28	Base value: 5	Cation half-life set to 0 to represent no degradation. Values for benzene and naphthalene determined through calibration.

Note: NA = not applicable

B2-3.3 Results of Initial Modeling

The model-predicted cation concentration is in general agreement with the average measured cation depth profile (Figure B2-2). The model results generally reproduce the pattern of decreasing cation concentration as the porewater nears the surface, but slightly underestimates the cation concentration in the 0 to 10 cm depth. The target normalized cation concentration for this depth is 0.57 +/- 0.15, while the model results predict an average concentration of 0.42; this would indicate that the effect of physical processes related to dispersive mixing (including bioturbation) and exchange with the surface water have been overestimated. Reducing some of the mixing related coefficients can produce a better match, for example reducing the porewater biodiffusion coefficient to a range more appropriate for a less dynamic setting, such as a freshwater lake (approximately 100 cm^2/yr), produces an average concentration of 0.58; however, using physical mixing parameter values that overestimate the reduction of cation concentration in the sediment column will allow for conservative values of the degradation rates to be generated in the subsequent benzene and naphthalene calibration.

The best-estimate values for surface exchange coefficient , dispersivity, and biodiffusion were retained in the cation model and then used in the model calibration for benzene and naphthalene. For the naphthalene calibration a range of groundwater seepage velocities were used, in addition to the base value of 176 cm/yr for nearshore areas. To reproduce the measured porewater benzene and naphthalene concentration profiles, use of non-zero degradation rates in the model was required; this was achieved by using the previous values for dispersive mixing and surface exchange from the cation model, modifying fixed chemical-specific coefficients and adjusting the degradation rates for benzene and naphthalene to calibrate.

Degradation rates for benzene and naphthalene estimated through the calibration process are represented by half-life values of 5 days and 7 days (range of 7 to 36 days for sensitivity cases), respectively. As shown on Figure B2-3, the modeled concentration profiles of naphthalene generally fit the measured values, although porewater concentration are slightly overestimated (a target of 1.19 µg/L in the 0 to 10 cm layer, and model prediction of 1.89 µg/L). Recognizing that these values are on the low-end (higher degradation rate) of literature-based (Chung and King, 1991 and Heitkamp, et. al., 1987) values for half-lives (but are not out of the range of what has been observed), the decision was made not to further decrease the half-lives to force a better fit. Due to suspicion that possible overestimation of the groundwater seepage lead to overestimation of degradation rates, a range of calibrated degradation rates corresponding to a range of input groundwater seepage (range 66 to 176 cm/yr) were computed (shown in Figure B2.4). All the seepage rate/degradation rate combinations resulted in an average porewater naphthalene concentration in the 0 to 10 cm layer of approximately 1.9 µg/L.

Even with a slight overestimation of the physical mixing related reduction in concentration, as noted in the cation simulation, without degradation, the benzene and naphthalene models would substantially over-predict (by a factor of 20) the benzene and naphthalene concentrations measured in the porewater near the sediment surface. The difference in the magnitude of cation (approximately a 50 percent reduction) and COC concentrations (approximately a 99 percent reduction) decline as they approach the surface provides strong evidence that reduction in contaminant concentration is much more than simple mixing and dilution with surface water, and that contaminant degradation must be occurring in the sediment.

B2-2 Capping Evaluation

B2-4.1 Model Application Approach

Following the calibration process described in Section B2-3, the UT model was used to assess the performance of the chemical isolation component of the engineered sand cap included in the remedial alternatives (Figure 6-1 of main FS report), taking into account the conditions expected in this area (i.e., cap thicknesses and groundwater seepage velocities). Long-term cap performance was assessed by its ability to meet the following PRGs developed for the Site:

- **1.1 µg/L naphthalene**, based on the conservative ecological screening value developed by EPA Regions 3 and 5. As discussed in Sections 4.3.4 and 7 of the main FS text, the polycyclic aromatic hydrocarbon (PAH) equilibrium screening-level benchmark quotient (ESBQ) applied per EPA guidance, builds on the results of the baseline risk assessment and provides a more accurate determination of the protectiveness of alternative sediment cleanup actions.
- **22 µg/L benzene**, based on the National Water Quality Criteria for human health (water + organisms).

These model evaluations accounted for the effects of upland hydraulic controls and constructed caps under the wide range of remedial alternatives evaluated in this FS. To simplify the assessment, the model input parameters were selected by using conservative values to represent the range of FS alternatives.

B2-4.2 Model Setup and Inputs

B2-4.2.1 Model Domain and Layers

The preliminary engineered sand cap design evaluated for the nearshore sediment area consists of a bioturbation layer (8 cm) over a chemical isolation layer (approximately 1.25 feet sand), which would be placed over native sediment. An erosion protection layer would be required in the nearshore and bank sediment areas. Any added benefit provided by the erosion protection layer in reducing COC migration from the cap is not included in this evaluation. Only the bioturbation and chemical isolation layers were modeled for this FS. Therefore, the cap profile simulated in the model for the nearshore area of the Site consists of a total 1.5-feet (45.7 cm.) sand layer.

B2-4.2.2 Model Input Parameters

Most of the input parameters used for the capping simulations were the same as those developed from the current conditions modeling, as described in Section B2-3.2 and listed on Table B2-3. The only inputs that differed were the initial porewater concentration (boundary condition) at the base of the cap, and those necessary to simulate the remedial alternatives, which included the thickness and properties of the cap and the groundwater seepage velocity achieved following upland hydraulic controls. These inputs are described in detail in the sections that follow.

B2-4.2.2.1 Initial Porewater Concentrations

The measured surface sediment (0 to 10 cm) porewater concentrations from Table B2.2 were used for model inputs representing the porewater concentration entering the bottom boundary of the cap. The values are 0.46 µg/L for benzene and 1.19 µg/L for naphthalene.

B2-4.2.2.2 Groundwater Seepage Velocity

As discussed previously, results from the groundwater flow model were used to calculate the average groundwater seepage velocities in the nearshore and offshore areas (Table A-8, in Appendix A). The magnitude of the estimated groundwater velocities was dependent on the distance from shore and the remedial alternative selected for the modeling evaluation. In the nearshore area, the average predicted groundwater velocity is 147 cm/yr when upland caps are considered[1].

B2-4.2.2.3 Type of Material

Sand is used for cap material; therefore, for the cap material type in the model "G", indicating granular, was used. Even though the model was calibrated on native sediments composed of silts and clay, the model can be readily used to simulate granular cap material performance since the only term in the model that is affected by the material type is the effective diffusion coefficient. As observed by the differences between cation calibration and COC (benzene and naphthalene) calibration, for COCs the bigger driver for contaminant reduction is not diffusion but degradation. All the other parameters are same as calibrated values.

B2-4.2.2.4 Model Input Summary

The complete set of input values used in the capping evaluations, including those described above, is provided in Table B2-4. The inputs are divided into the following two categories based on the processes they characterize:

- Cap properties, which include physical properties of the evaluated capping material; and
- Chemical-specific properties.

[1] Predicted Darcy discharge velocities for the groundwater model runs representative of an upland capping alternative were used in offshore and nearshore sediment cap modeling; therefore, additional flux reductions provided by funnel and gate system hydraulic controls were not included in the model.

Table B2-4 – Cap Modeling Input Parameters Used in the Capping Evaluation

Model Input Parameters	Value		Notes
Porosity, e	0.4		Porosity of coarse sand (0.4).
Cap Materials - Granular (G)	G		Based on anticipated cap material type, this input was specified as “Granular material (G)”, which causes the model to calculate the effective molecular diffusion coefficient as a function of porosity based on the formulation of Millington and Quirk (1961).
Darcy Velocity, Vdar (positive is upwelling) in cm/yr	Nearshore: 147.1		Average groundwater seepage velocities representative of simulated conditions for each area and alternative based on the Site groundwater flow model (Table A-8, in Appendix A).
Particle Density, ρ_P in g/cm^3	2.5		Typical value for sand particles (e.g., Domenico and Schwartz 1990).
Biological Active Zone fraction organic carbon, $(f_{oc})_{bio}$	8%		Average value from top 10 cm of the sediments at the Site.
Fraction organic carbon, $(foc)_{eff}$	0.1%		Nominal value for sand cap.
Dispersivity, α in cm	4.57		Percent value determined through calibration to average near-shore cation concentrations (10% of model domain length).
Cap thickness in cm	45.7		Sand cap thickness
Chemical-Specific Parameters	**Naphthalene**	**Benzene**	**Notes**
Contaminant Initial Porewater Concentration, C_0 in µg/L	Nearshore: 1.19	0.46	Porewater concentrations represent average values from top 10 cm.

B2-4.3 Results of Cap Modeling Evaluation

The results of the cap chemical transport modeling indicate that the cap evaluated for the nearshore area of the Site, as described previously in Section B2-4.2.1 (i.e., 1.5 feet of sand), is predicted to achieve the PRGs at steady-state. This is not surprising given that the current average porewater concentration in the sampled 0 to 10 cm layer is already near or below the respective PRGs for naphthalene and benzene. The model simulated concentration profile of naphthalene in the cap is presented in Figure B2-5. The model computed concentrations in the upper-portion of the cap (expressed as the concentration of porewater entering the bottom of the bioturbation layer [8 cm] and the vertical averages over the top 10 cm [representing the sampled depth]) were compared to current surface concentrations and PRGs in Table B2-5, and are summarized below.

In the nearshore area, benzene and naphthalene concentrations in the top 10 cm of the cap are predicted to be nearly 100 times less than the current porewater concentrations in the surface sediment. The average porewater concentrations at the base of the bioturbation layer (8 cm depth) are predicted to be for naphthalene more than 50 times less and for benzene more than 100 times less than the current porewater concentrations in the surface sediment. For both depths, the predicted concentrations are well below the PRGs, by factors of an order of magnitude or more.

Table B2-5 – Model-Predicted Vertical Average Concentrations for Cap Evaluation

Modeled Area	Chemical	PRG in μg/L[1]	Current Surface (0-10 cm) Porewater Concentration in μg /L	Model-Predicted Average Concentration in μg /L	
				at 8 cm	0-10 cm
Nearshore (1.5-foot sand cap)	Naphthalene	1.1	1.19	0.017	0.012
	Benzene	22	0.46	0.0032	0.0026

Note:
[1] PRG for naphthalene is based on ecological risk criteria. PRG for benzene is based on a human health standard.

B2-4.4 Sensitivity Analyses

Several of the model input parameters have uncertainty/variability associated with them, such as initial COC concentrations, groundwater seepage velocity, degradation rate, and the physical attenuation parameters that were calibrated (i.e., dispersion and Kbl).

The porewater concentrations computed at various depths are linearly a function of the initial concentration specified; doubling the initial concentration doubles the computed concentration at all depths. Given the reduction in relative porewater concentration determined by the model, initial porewater concentrations at the sediment/cap interface could be 106 μg/L for naphthalene and 3,900 μg/L for benzene, and the concentrations in the 0 to 10 cm layer would still meet the respective PRG.

Given the very low initial concentration of benzene in porewater compared to the PRG, only naphthalene was used in the sensitivity analyses. The model input parameter sets used in these sensitivity analyses and the results of the sensitivity analyses are listed in Table B2-6.

Table B2-6 – Sensitivity Analyses Input Parameters and Results

	Naphthalene Concentration (μg/L)	
	avg. 0 -10 cm	**at 8 cm**
Base Model	0.0124	0.0171
Seepage Velocity		
147 cm/yr (base)	0.0124	0.0171
100 cm/yr	0.0036	0.0048
200 cm/yr	0.0288	0.0397
300 cm/yr	0.0707	0.0981
Boundary Layer Mass Transfer Coefficient, Kbl		
0.33 cm/hr (base)	0.0124	0.0171
0.2 cm/hr	0.0131	0.0176
0.5 cm/hr	0.0119	0.0169
1 cm/hr	0.0114	0.0164
Dispersivity, α		
4.57 cm (base; 10%)	0.0124	0.0171
2.28 cm (5%)	0.0071	0.0093
6.85 cm (15%)	0.0169	0.0252
9.14 cm (20%)	0.0236	0.0336
Degradation Half-life		
7 days (base)	0.0124	0.0171
14 days	0.0514	0.0709
21 days	0.0888	0.1223
28 days	0.1197	0.1649
Cap Thickness		
45.7 cm (base; 1.5 ft)	0.0124	0.0171
40 cm	0.0193	0.0257
30 cm	0.0445	0.0600
Bioturbation depth		
8 cm (base)	0.0124	0.0171
4 cm	0.0155	0.0267
12 cm	0.0131	0.0195
Porosity		
40% (base)	0.0124	0.0171
30%	0.0233	0.0319
50%	0.0071	0.0098
Porewater Biodiffusion (and particle biodiff *100)		
900 cm^2/yr (base)	0.0124	0.0171
100 cm^2/yr	0.0183	0.0271
300 cm^2/yr	0.0162	0.0236
1,800 cm^2/yr	0.0100	0.0129

Compared to current surface naphthalene porewater concentrations, the model results for sensitivity cases are still at least 10 times lower in all instances. Similarly, compared to the PRGs, the results from the sensitivity simulations based on alternate parameter sets are generally 10 times lower than the PRGs.

Most of the parameters used in the sensitivity analysis exhibited relative low influence, especially the results in comparison to the PRG. This may be due to the concentration reductions observed being more a factor of degradation rather than sorption reactions with the cap material. The three input parameters which exhibited the most influence were the cap thickness, the seepage velocity, and the degradation half-life. These parameters are related in that they determine how many half-lives the COCs will remain within the cap. The cap thickness and seepage velocity are fundamental in the determination of the residence time of the chemical within the cap, while the degradation half-life determines the rate at which the chemical breaks down.

As noted earlier in the COC calibration, there is an interdependency of groundwater seepage flux with degradation. In the calibration, as one increases the other follows. Various combinations of groundwater seepage and degradation yielding acceptable calibrations were developed and these were then used in capping scenarios. When considered together the individual effects of groundwater seepage velocity and degradation rates are significantly reduced, indicating that these two parameters each may have uncertainty, and calibrating them together to a Site-specific concentration profile reduces the overall modeling generated variability (Table B2-7).

Table B2-7 – Sensitivity for Paired Calibration of Seepage Velocity and Degradation Rate

Darcy Velocity (cm/yr)	Degradation Half-Live (days)	Napthphalene Concentration (µg/L)	
		at 8 cm	0-10 cm average
55	36	0.038	0.03
73	21	0.027	0.021
92	14	0.024	0.017
110	10.5	0.02	0.015
125	8.8	0.019	0.014
147	7	0.017	0.012

B2-5 References for Appendix B2

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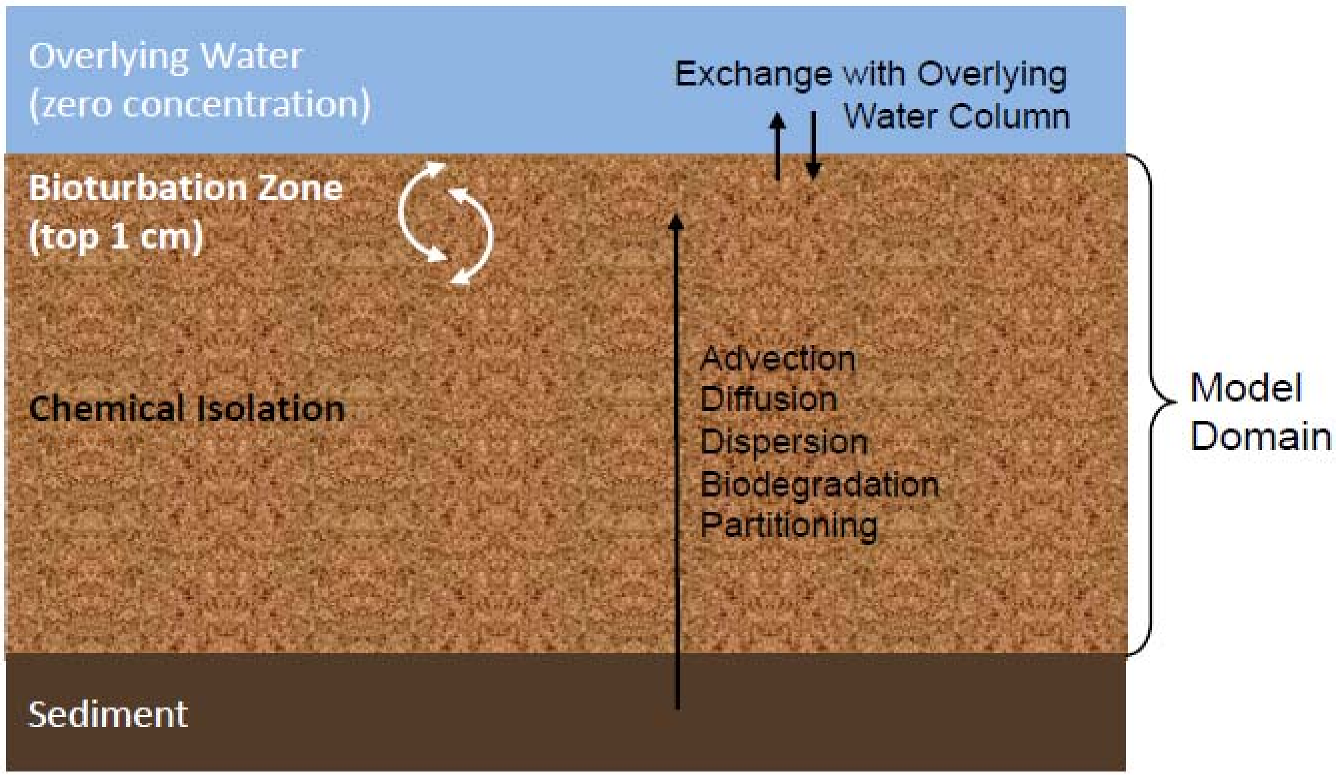

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Figure B2-1
Schematic of Model Configuration and Processes
Quendall Terminals Feasibility Study Report
Renton, Washington

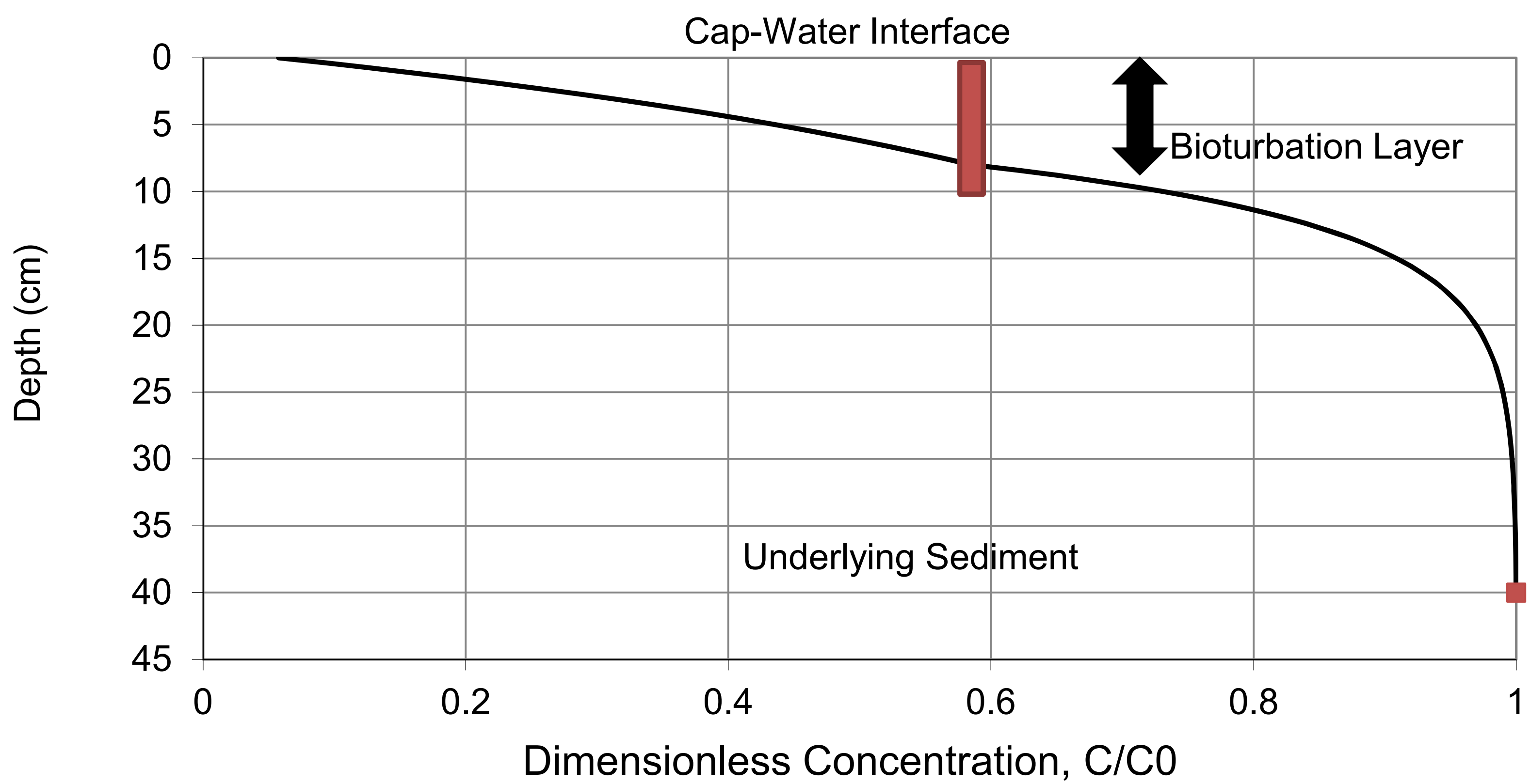

Figure B2-2
Cation Normalized Concentration Profile in Current Sediment Conditions
Quendall Terminals Feasibility Study Report
Renton, Washington

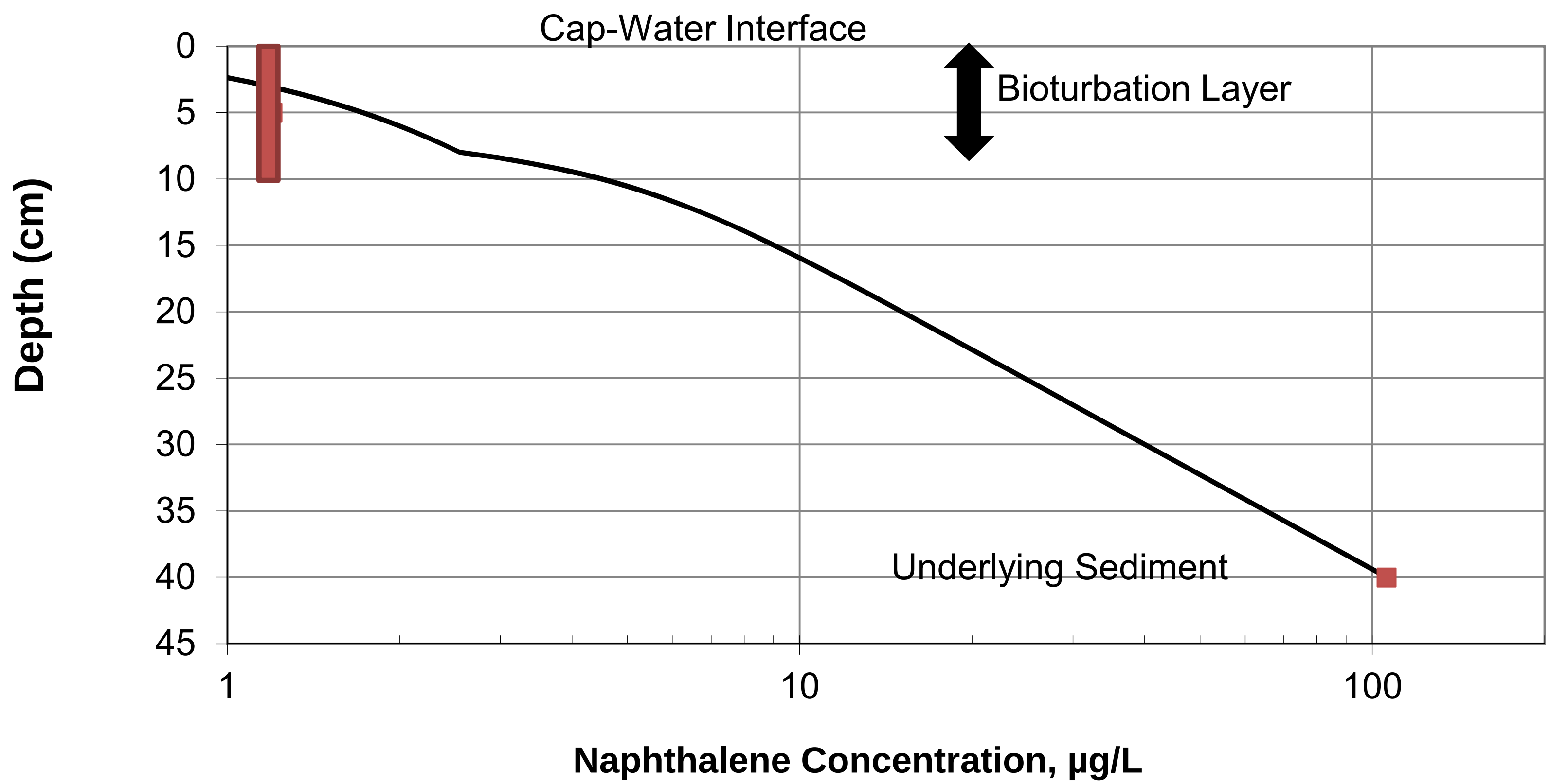

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Figure B2-3
Naphthalene Concentration Calibration to Current Sediment Conditions
Quendall Terminals Feasibility Study Report
Renton, Washington

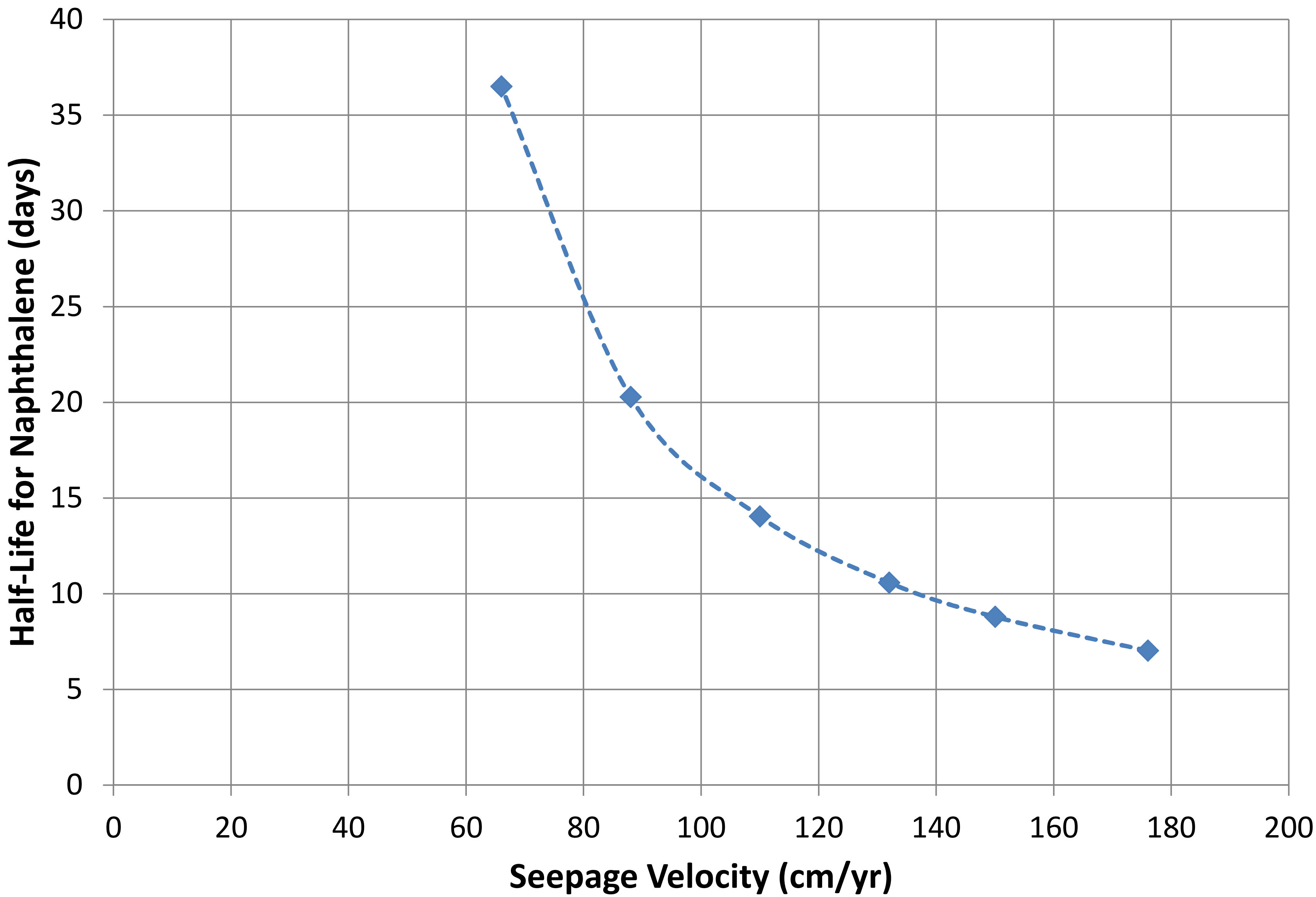

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Figure B2-4

Naphthalene Calibration - Relationship of Degradation Half-life to Seepage Velocity

Quendall Terminals Feasibility Study Report

Renton, Washington

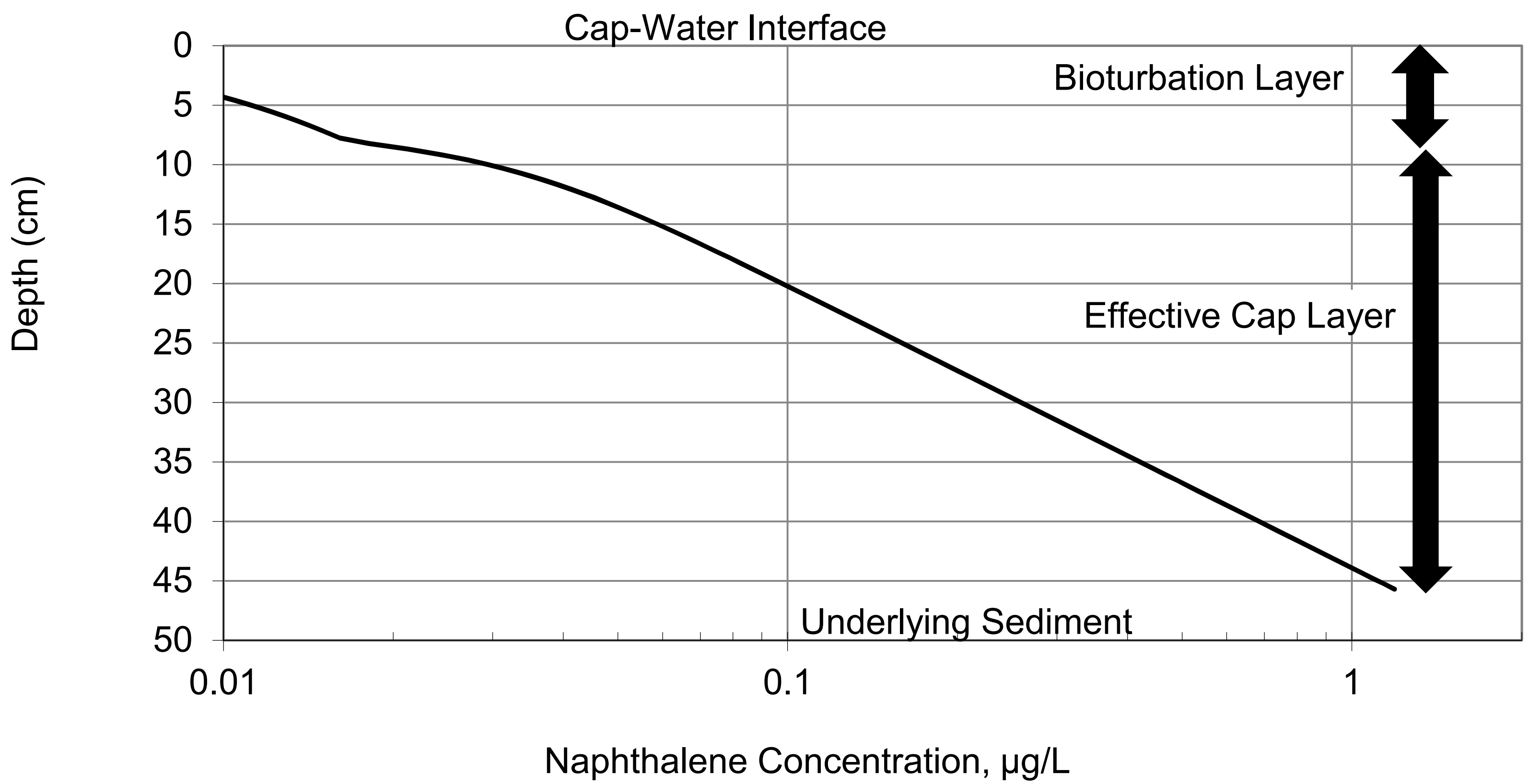

Figure B2-5
Simulated Naphthalene Concentration Profile in Cap
Quendall Terminals Feasibility Study Report
Renton, Washington

Attachment 1 of Appendix B2 - Sediment Porewater Analytical Data

Quendall Terminals
Renton, Washington

Sample Location	Sample Name	Date	Magnesium µg/l	Sodium µg/l	Calcium µg/l	Potassium µg/l	Benzene µg/l	Naphthalene µg/l
NS-04-SS	NS-04-PW	6/16/2009	--	--	--	--	< 0.2 U	< 0.83 U
NS-04-SS	NS-04-PW-0-4	6/16/2009	5650	7625	16250	2875	--	--
NS-04-SS	NS-04-PW-8-12	6/16/2009	7645	15730	26950	4950	--	--
NS-04-VC	NS-04-VC-0-4	6/25/2009	--	--	--	--	< 1 U	< 0.83 U
NS-04-VC	NS-04-VC-12-16	6/25/2009	--	--	--	--	--	--
NS-04-VC	NS-04-VC-20-24	6/25/2009	--	--	--	--	4	2.23
NS-04-VC	NS-04-VC-28-32	6/25/2009	10509	20340	18758	2260	--	--
NS-04-VC	NS-04-VC-36-40	6/25/2009	--	--	--	--	2.6	0.844
NS-04-VC	NS-04-VC-44-48	6/25/2009	15504	23712	27360	3078	--	--
NS-04-VC	NS-04-VC-8-12	6/25/2009	--	--	--	--	< 1 U	< 0.83 U
NS-05-SS	NS-05-PW	6/16/2009	--	--	--	--	< 1 U	0.974
NS-05-SS	NS-05-PW-0-4	6/16/2009	6815.9	6741	23326	4280	--	--
NS-05-SS	NS-05-PW-8-12	6/16/2009	9590.4	10692	38664	5292	--	--
NS-05-VC	NS-05-VC-0-4	6/25/2009	--	--	--	--	1.5	332 J
NS-05-VC	NS-05-VC-20-24	6/25/2009	--	--	--	--	19	17.6
NS-05-VC	NS-05-VC-28-32	6/25/2009	5702.8	14734	10918	2226	--	--
NS-05-VC	NS-05-VC-36-40	6/25/2009	--	--	--	--	< 1 U	7.01
NS-05-VC	NS-05-VC-44-48	6/25/2009	6741	15836	11984	2461	--	--
NS-05-VC	NS-05-VC-48-60	6/25/2009	--	--	--	--	< 1 U	0.568 J
NS-05-VC	NS-05-VC-8-12	6/25/2009	--	--	--	--	--	844 J
NS-05-VC	NS-55-VC-48-60	6/25/2009	--	--	--	--	0.6 J	2.53
NS-06-SS	NS-06-PW	6/17/2009	--	--	--	--	< 0.2 U	< 0.83 U
NS-06-VC	NS-06-VC-0-4	6/30/2009	--	--	--	--	< 1 U	< 0.83 U
NS-06-VC	NS-06-VC-20-24	6/30/2009	--	--	--	--	< 1 UJ	< 0.83 U
NS-06-VC	NS-06-VC-8-12	6/30/2009	--	--	--	--	< 1 UJ	< 0.83 U
NS-07-SS	NS-07-PW	6/16/2009	--	--	--	--	< 0.2 U	< 0.83 U
NS-07-SS	NS-07-PW-0-4	6/16/2009	6684.7	7004	19982	1339	--	--
NS-07-SS	NS-07-PW-8-12	6/16/2009	8137	11639	21630	1751	--	--
NS-07-VC	NS-07-VC-0-4	6/30/2009	--	--	--	--	< 1 U	< 0.83 U
NS-07-VC	NS-07-VC-20-24	6/30/2009	--	--	--	--	--	< 0.83 U
NS-07-VC	NS-07-VC-28-32	6/30/2009	19470	26070	42460	4070	--	--
NS-07-VC	NS-07-VC-36-40	6/30/2009	--	--	--	--	--	< 0.83 U
NS-07-VC	NS-07-VC-44-48	6/30/2009	22230	24570	46917	4680	--	--
NS-07-VC	NS-07-VC-8-12	6/30/2009	--	--	--	--	< 1 U	< 0.83 U
NS-08-SS	NS-08-PW	6/17/2009	--	--	--	--	2.1	4.2
NS-08-SS	NS-08-PW-0-4	6/17/2009	21527	16789	41715	2060	--	--
NS-08-SS	NS-08-PW-8-12	6/17/2009	21726	17748	42738	2244	--	--
NS-08-VC	NS-08-VC-0-4	6/29/2009	--	--	--	--	5.7	2.5
NS-08-VC	NS-08-VC-20-24	6/29/2009	--	--	--	--	1000	5.73
NS-08-VC	NS-08-VC-28-32	6/29/2009	28200	24000	53280	3600	--	--
NS-08-VC	NS-08-VC-36-40	6/29/2009	--	--	--	--	--	8.36
NS-08-VC	NS-08-VC-44-48	6/29/2009	22815	24219	43524	3744	--	--
NS-08-VC	NS-08-VC-8-12	6/29/2009	--	--	--	--	1200	6.34
NS-09-SS	NS-09-PW	6/17/2009	--	--	--	--	< 0.2 U	2.26
NS-09-SS	NS-09-PW-0-4	6/17/2009	6252.1	6386	20394	2369	--	--
NS-09-SS	NS-09-PW-8-12	6/17/2009	5050.2	9120	12996	2850	--	--
NS-09-SS	NS-59-PW	6/17/2009	--	--	--	--	--	< 0.83 U
NS-09-VC	NS-09-VC-0-4	6/24/2009	--	--	--	--	140	2.6
NS-09-VC	NS-59-VC-0-4	6/24/2009	--	--	--	--	--	830 J
NS-12-SS	NS-12-PW	6/15/2009	--	--	--	--	< 0.2 U	< 0.83 U
NS-12-SS	NS-12-PW-0-4	6/15/2009	4091.6	5936	12084	1272	--	--
NS-12-SS	NS-12-PW-8-12	6/15/2009	5200.2	10593	15408	1605	--	--
NS-12-VC	NS-12-VC-0-4	6/29/2009	--	--	--	--	< 1 U	1.45
NS-12-VC	NS-12-VC-20-24	6/29/2009	--	--	--	--	--	< 0.83 U
NS-12-VC	NS-12-VC-36-40	6/29/2009	--	--	--	--	--	< 0.83 U
NS-12-VC	NS-12-VC-44-48	6/29/2009	9711	18252	17784	3276	--	--
NS-12-VC	NS-12-VC-8-12	6/29/2009	--	--	--	--	< 1 U	0.719 J
NS-13-SS	NS-13-PW	6/16/2009	--	--	--	--	< 0.2 U	< 0.83 U
NS-13-SS	NS-13-PW-0-4	6/16/2009	6386	6798	17407	1339	--	--
NS-13-SS	NS-13-PW-8-12	6/16/2009	9548.1	11948	21836	1854	--	--
NS-13-VC	NS-13-VC-0-4	6/25/2009	--	--	--	--	< 1 U	< 0.83 U
NS-13-VC	NS-13-VC-20-24	6/25/2009	--	--	--	--	48	< 0.83 U
NS-13-VC	NS-13-VC-28-32	6/25/2009	13589	17655	27499	2889	--	--
NS-13-VC	NS-13-VC-36-40	6/25/2009	--	--	--	--	< 1 U	< 0.83 U
NS-13-VC	NS-13-VC-44-48	6/25/2009	15260	20056	29103	3706	--	--
NS-13-VC	NS-13-VC-8-12	6/25/2009	--	--	--	--	2.2	< 0.83 U
NS-14-SS	NS-14-PW	6/17/2009	--	--	--	--	< 1 U	
NS-14-SS	NS-14-PW-8-12	6/17/2009	3619	4290	16940	1650	--	--
NS-14-VC	NS-14-VC-8-12	6/24/2009	--	--	--	--	--	< 0.83 U

Notes:
1. These are the sediment samples between the inner harborline and the shoreline and outside the DNAPL areas.
2. Only the subsurface sample which have corresponding surface sample data are presented here and used in model inputs.

APPENDIX B3

Cap Armor Layer Evaluation

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List of Tables

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B3-1 Introduction

This appendix summarizes the engineering evaluations conducted to develop a preliminary armor layer design that would promote long-term stability of a sediment isolation cap constructed at the Quendall Terminals Site (Site). The armor layer is intended to protect the chemical isolation layer and underlying contaminated sediments from erosional processes such as waves and propeller wash.

B3-2 Methodology

Screening-level analyses were performed to determine the required particle size and thickness for the sediment cap armor layer to resist erosive forces. Long-term wind data from a nearby wind gage was used to estimate various storm event return periods for the area from a variety of wind directions. These extreme wind speeds, fetch lengths, and average depths were then used to estimate the wave action that will influence the Site. Vessel-induced waves and propeller-wash forces were also evaluated. Predicted wave heights were used to estimate stable rock sizes for the potential cap areas as a function of water depth.

Engineering evaluations were conducted in accordance with guidance developed by the U.S. Army Corps of Engineers (USACE). In addition, the U.S. Environmental Protection Agency's (EPA) Contaminated Sediment Remediation Guidance for Hazardous Waste Sites (EPA 2005) states that, *"[t]he design of the erosion protection features of an in-situ cap (i.e., armor layers) should be based on the magnitude and probability of occurrence of relatively extreme erosive forces estimated at the capping site. Generally, in-situ caps should be designed to withstand forces with a probability of 0.01 per year, for example, the 100-year storm."*

B3-3 Analysis of Wave Action and Propeller Wash

B3-3.1 Water Levels

The elevation of Lake Washington is controlled by the Lake Washington Ship Canal, which connects Lake Washington to Lake Union and Puget Sound. As a result, ordinary low and ordinary high water lake elevations are 16.67 and 18.67 feet NAVD88, respectively, for this portion of Lake Washington.

B3-3.2 Evaluation of Wind-Induced Waves

The wave conditions near the Site were estimated by applying wind wave growth formulas to wind data from Sea-Tac International Airport (Sea-Tac) in Seattle, Washington (NOAA, WBAN #24233). Data were obtained through the National Climatic Data Center (http://www.ncdc.noaa.gov/oa/ncdc.html) for the time period of interest. The wind data encompassed hourly wind speeds (2-minute averages) between the years of 1990 and 2011. Figure B3-1 illustrates a wind rose (frequency of occurrence based on wind speed and wind direction) for the wind data over the period of record. The wind data were used to predict extreme wind speed values for 2-, 10-, 20-, 50-, and 100-year return period storm events. The extreme wind speeds were evaluated for 10-degree and 30-degree wind direction bins from true north (e.g., 0 to 10 degrees, 211 to 240 degrees, etc.) that impact the area. The Raleigh distribution was used to develop the extreme wind speeds with R^2 values equal to or greater than 0.87 for all direction bins.

Fetch lengths were measured for each wind directional zone that has the potential for wind waves to develop and impact the shoreline. Fetch measurements were completed based on methodology outlined in the CEM (USACE 2002). These fetch lengths and associated directions are summarized in Table B3-1.

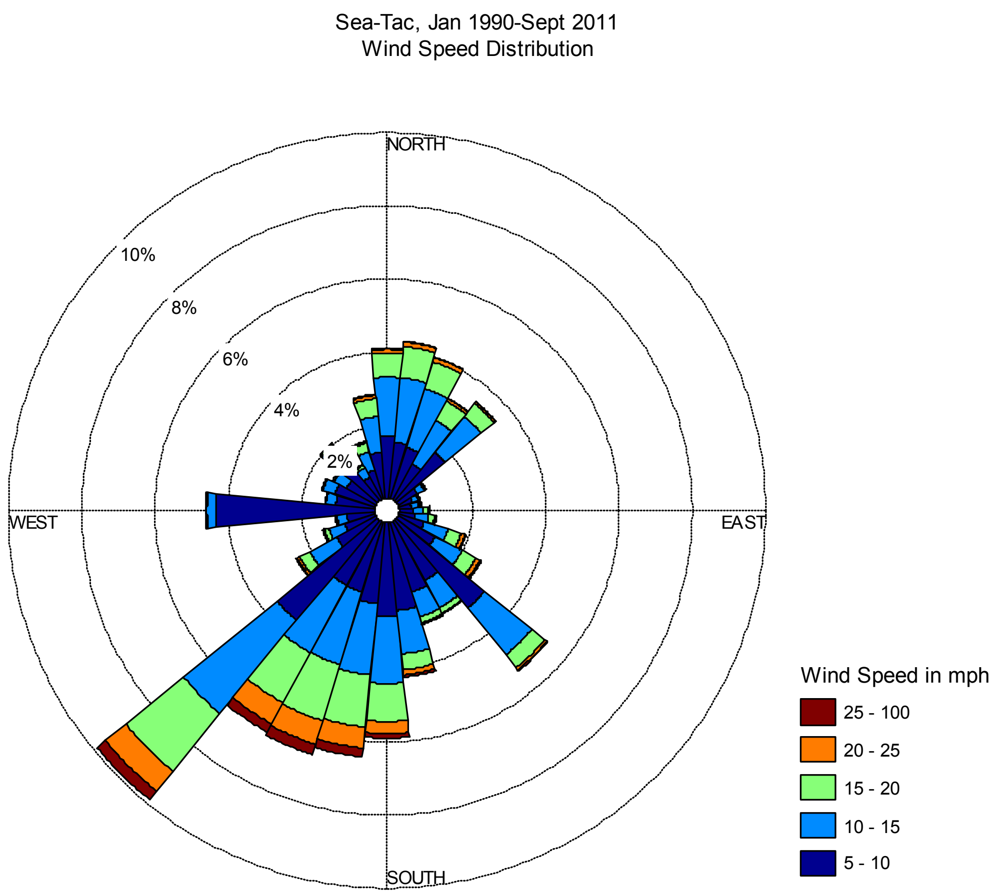

Figure B3-1 – Sea-Tac Wind Speed Distribution (January 1990–September 2011)

Predicted values of wind speed for a range of return periods were used as input into the Automated Coastal Engineering System (ACES) using the Windspeed Adjustment and Wave Growth module (fetch limited) to predict significant wave heights and peak wave periods generated by the extreme winds (USACE 1992). Results of the wave growth analysis are shown in Table B3-2. The highest winds and waves are from the southwest (as shown on Figure B3-1 and in Table B3-1). During a 100-year storm from the southwest, waves are estimated to be 3.5 feet high. Waves from the north (331 to 10 degrees) are also high based on high winds and long fetches. During a 100-year storm from 331 to 360 degrees waves heights are estimated to be 2 feet and from 0 to 10 degrees they are expected to be approximately 1.7 feet.

Table B3-1 – Wind Speeds and Fetch

Wind Direction Zone	Fetch Length in Miles	Water Depth in Feet[1]	Wind Speed as a Function of Return Period in mph				
			2-year	10-year	20-year	50-year	100-year
0 to 10 deg	3.1	60	25	29	31	32	34
11 to 20 deg	1.1	70	25	29	31	33	35
21 to 30 deg	0.7	50	23	28	30	32	33
211 to 240 deg	2.3	90	37	48	52	58	60
241 to 270 deg	0.8	60	25	32	35	38	40
271 to 300 deg	0.5	70	17	22	24	25	27
301 to 330 deg	0.5	60	20	30	32	36	38
331 to 360 deg	1.5	60	28	37	40	44	46

Notes:
1. Average water depth at location where wave is generated (i.e., over the fetch length).

Table B3-2 – Predicted Wave Height and Period

Wind Direction Zone	Return Period									
	2-year		10-year		20-year		50-year		100-year	
	Wave Height in Feet	Wave Period in Seconds	Wave Height in Feet	Wave Period in Seconds	Wave Height in Feet	Wave Period in Seconds	Wave Height in Feet	Wave Period in Seconds	Wave Height in Feet	Wave Period in Seconds
0 to 10 deg	1.1	2.0	1.3	2.2	1.4	2.3	1.5	2.3	1.6	2.4
11 to 20 deg	0.6	1.5	0.8	1.6	0.8	1.7	0.9	1.8	1.0	1.8
21 to 30 deg	0.4	1.3	0.6	1.4	0.6	1.5	0.7	1.5	0.7	1.6
211 to 240 deg	1.8	2.5	2.4	2.9	2.7	3.0	3.3	3.3	3.5	3.4
241 to 270 deg	0.6	1.5	0.9	1.7	1.0	1.8	1.1	1.9	1.2	2.0
271 to 300 deg	0.3	1.1	0.4	1.2	0.5	1.3	0.5	1.3	0.5	1.4
301 to 330 deg	0.4	1.2	0.6	1.5	0.7	1.5	0.8	1.6	0.9	1.7
331 to 360 deg	1.0	1.9	1.5	2.2	1.6	2.	1.8	2.5	2.0	2.5

B3-3.3 Evaluation of Vessel-Induced Waves

A systematic vessel study has not been completed for this evaluation. However, based on Site knowledge it is anticipated that the project aquatic and shoreline areas will be impacted by wakes from passing recreational boats operating offshore in Lake Washington adjacent to the Site.

Design wave heights resulting from wind waves (Section B3-3.2) are expected to be higher than wakes for the Site. To verify this assumption, wake heights were calculated for a representative high performance recreational boat for various vessel speeds at various distances from the project shoreline. Characteristics of this representative vessel are summarized below:

Type of Vessel: Baja Outlaw 23

Propeller Shaft Depth: 2.75 feet

Number of Engines: 1

Engine Horsepower: 375

Propeller Dimensions: 17 inches

This vessel represents a reasonable worst case scenario within Lake Washington for both wake and propeller-wash velocities at the Site, and has been used for similar evaluations at other sites (Parsons and Anchor QEA 2012). If capping is selected as a final remedy at the Site, a more robust vessel survey would be conducted for the project area during remedial design to refine this evaluation in the design phase for this project.

Wake heights were calculated using an analytical method developed by Bhowmik et al. (1991). This method is based on empirical data from 12 different recreational type vessels and is applicable for recreational vessels operating at a speed of between 8 and 45 miles per hour (Bhowmik et al. 1991, Parsons and Anchor QEA 2012). Wake heights were estimated for the representative design vessel over a range of operating speeds and offshore passing distances. Computed wake heights ranged from 0.5 foot to a maximum of 2.2 feet (for a vessel passing 10 feet offshore of the Site). As anticipated, these wake heights are less than the maximum wave height estimated for wind-induced waves (Table B3-2). Therefore, the wind-induced waves were used in the analysis.

B3-3.4 Evaluation of Propeller-Wash Velocities

Proposed caps in deeper water away from the shoreline (water depths greater than 5 feet) may be subject to propeller-induced velocities that will be greater than those created by wind- and vessel-induced waves. Therefore, propeller-wash velocities in these capping areas may be the dominant factor in sizing stable cap material.

To estimate the bed velocity resulting from propeller wash, the Blaauw and van de Kaa (1978) method was used with the characteristics of the design vessel (described in Section B3-3.3).

$$V_b(max) = C_1 U_o D_p / H_p$$

Where:

$V_b(max)$	=	maximum bottom velocity in ft/sec
C_1	=	0.22 for non-ducted propeller
	=	0.30 for ducted propeller
U_o	=	jet velocity exiting propeller in ft/sec
D_p	=	propeller diameter in feet
H_p	=	distance from propeller shaft to channel bottom in feet

The jet velocity exiting a propeller is given by Blaauw and van de Kaa (1978) as

$$U_o = C_2 \left(\frac{P_d}{D_p^2}\right)^{1/3}$$

Where:

U_o	=	jet velocity exiting propeller in ft/sec
P_d	=	applied engine power/propeller in Hp
D_p	=	propeller diameter in ft
C_2	=	9.72 for non-ducted propellers
C_2	=	0.68 for ducted propellers

Propeller-wash velocities at the bed for various water depths associated with the proposed capping areas were calculated using the equations above and are summarized in Table B3-3.

Table B3-3 – Maximum Predicted Bed Velocities from Propeller Wash for Various Water Depths

	Applied Engine Power from Design Vessel (Section B3-2.2)			
Water Depth based on Low Lake Level in Feet	**85%**	**75%**	**50%**	**25%**
	Maximum Predicted Bed Velocity in ft/sec			
5.5	4.1	3.9	3.4	2.7
14.5	1.1	1.1	1.0	0.8
16.5	1.0	0.9	0.8	0.7
21.5	0.7	0.7	0.6	0.5
25.5	0.6	0.6	0.5	0.4

B3-4 Armor Size Evaluation

B3-4.1 Cap Armor Size – Breaking-Wave Zone

The ACES Rubble Mound Revetment Design module was used to estimate revetment armor and bedding layer stone sizes, thicknesses, and gradation characteristics required; as well as runup estimates (USACE 1992). Table B3-4 provides the median (D_{50}) rock size that would be stable (limited to no damage) for the given waves in Table B3-2 for a slope of 10H:1V. Table B3-4 also provides the vertical runup height. The vertical runup represents the expected maximum runup using the Ahrens and Heimbaugh method (USACE 1992). The worst case is from direction 211 to 240 degrees with a 5.3-inch armor stone required for caps located within the breaking-wave zone defined in the next section.

Table B3-4 –Stable Armor Rock Size and Runup for a 10H:1V Slope within the Breaking-Wave Zone

Wind Direction Zone	Armor Size D_{50} in Inches	Runup Distance in Feet
0 to 10 deg	2.5	0.8
11 to 20 deg	1.6	0.5
21 to 30 deg	1.2	0.4
211 to 240 deg	5.3	1.6
241 to 270 deg	1.8	0.6
271 to 300 deg	0.8	0.3
301 to 330 deg	1.3	0.4
331 to 360 deg	3.0	0.9

B3-4.2 Cap Armor Extent – Breaking-Wave Zone

The cap armor along the shoreline should extend up slope to the vertical extent of wave runup based on the water level elevation at high water and down slope to a depth that is no longer impacted by the breaking waves at low water (i.e., the breaking-wave zone). The highest runup elevation is estimated by adding the runup height (shown in Table B3-4) to the elevation of ordinary high water at the Site (18.7 feet NAVD88). The lower bound of the armor is estimated by multiplying the significant wave height by 1.5 and subtracting that number for the low water elevation (approximately 16.7 feet NAVD88) (USACE 2002). The upper bound of the intertidal cap armor should be 19.3 feet NAVD88 and the lower bound of the armor should be 11 feet NAVD88 (16.7 feet low water minus 1.5 times the largest wave of 3.52 feet). This would correspond to a water depth of approximately 5.5 feet (based on the low water level).

B3-4.3 Cap Armor Size – Non-Breaking-Wave Zone

Armor stone blanket stability design (USACE 2002) was used to estimate the D_{50} required for the areas below the influence of breaking waves (i.e., approximately elevation 11 feet NAVD88). Gradation was calculated using HQUSACE 1994 method described in the Coastal Engineering Manual (USACE 2002). The proposed armor size is based on the worst case 100-year return period wind direction, which is 211 to 240 degrees (significant wave height is 3.5 feet).

Below the breaking-wave zone (11 feet NAVD88; approximately 5-foot water depth based on low water level) down to an elevation of approximately 1 foot NAVD88 (approximately 15-foot water depth based on low water level), the stable rock size is 0.6 inch. At elevations below 1 foot NAVD88, stable rock sizes are reduced to 0.06 inch (sand).

B3-4.4 Cap Armor Size – Propeller-Wash Zone

Methods presented in the USEPA guidance (Maynord 1998) to evaluate stable sediment size for propeller-wash velocities at the bed (Blaauw and van de Kaa 1978) are based on large ocean-going vessels operating at very slow speeds. Therefore, these methods are not applicable for use with smaller, fast-moving recreational vessels. A more robust analysis to evaluate stable sediment sizes for propeller wash from recreational vessels was conducted to inform capping design for the Fox River (Shaw and Anchor 2007) and Onondaga Lake (Parsons and Anchor QEA 2012) projects. Results from these previous studies were used to estimate stable sediment sizes for the range of bed velocities induced by propeller wash summarized in Table B3-3. Based on characteristics of the design vessel (Section B3-3.3), stable particle sizes for a range of water depths and applied horsepower is summarized in Table B3-5.

Table B3-5 – Stable Sediment Size below the Breaking-Wave Zone for Propeller-Wash Velocities

Water Depth in Feet (based on low water level)	Applied Horsepower in Percent	Median Particle Size (D_{50}) in Inches / Sediment Type
≤ 6	25	0.2 / coarse sand
	50	0.3 / fine gravel
	75	0.4 / fine gravel
	100	0.5 / fine gravel
≥10	25	0.01 / fine sand
	50	0.01 / fine sand
	75	0.01 / fine sand
	100	0.02 / medium sand

B3-5 Conclusions

The proposed capping areas extend from relatively deep water (> 15 feet) to shoreline areas at the Site. These areas are impacted by both wind- and vessel-induced waves and propeller-wash forces. The process that dominated the stable armor/sediment size evaluation is dependent on water depth (i.e., a D_{50} value from the breaking-wave evaluation will influence the stable particle size to a greater degree than propeller-wash forces in shallow water and vice versa in deeper water). Table B3-6 summarizes the recommended median (D_{50}) stable armor/sediment sizes at each water depth based on the above evaluations.

Table B3-6 – Recommended Armor D_{50} Values as Function of Water Depth (based on low water level of 16.7 feet NAVD88)

Water Depth in Feet (based on low water level)	Armor Size D_{50} in Inches	Dominant Process
≤ 5	6.0	Breaking Waves (Sections B3-3.1 and B3-3.2)
≤ 5 and ≥15	0.6	Non-Breaking Waves (Section B3-3.3)
≥15	0.01	Propeller Wash – 75% applied power (Section B3-3.4)

B3-6 References

Bhowmik, N., T. Soong, W. Reichelt, and N. Seddik, 1991, Waves Generated by Recreational Traffic on the Upper Mississippi River System, Illinois State Water Survey, Department of Energy and Natural Resources.

Blaauw, H.G., and E.J. van de Kaa, 1978, Erosion of Bottom and Sloping Banks Caused by the Screw Race of Maneuvering Ships. Paper presented at the 7th International Harbour Congress, Antwerp, Belgium. May 22-26, 1978

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Maynord, S., 1998, Appendix A: Armor Layer Design for the Guidance for In-Situ Subaqueous Capping of Contaminated Sediment. Prepared for the U.S. Environmental Protection Agency (EPA).

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APPENDIX B4

Cap Geotechnical Considerations

Contents

List of Tables

B4-1 Introduction

This appendix presents a preliminary feasibility-level evaluation of geotechnical considerations in conjunction with remedial alternatives for the Quendall Terminals Site (Site) that include subaqueous capping. These alternatives are discussed in Section 6 of the main text. This appendix provides discussions in regards to cap settlement, bearing capacity during cap construction, and seismic considerations. Conclusions regarding the overall feasibility of subaqueous capping at the Site, and design and construction considerations are provided at the end of this appendix.

B4-2 Subsurface Conditions

Subsurface conditions used for this analysis were based in part on a review of existing geotechnical engineering reports for the Site (Aspect 2009). Geotechnical borings logs and sediment core logs collected as part of the Remedial Investigation (RI) (Anchor QEA and Aspect 2012), as well as laboratory data, and historical geotechnical borings by others (Twelker and Associates 1973, Shannon and Wilson 1997) were also used in assessing subsurface conditions and properties. Figures 3-4, 4-2, 4-3, 4-4, and 4-5 of the main text show cross-sections of soil and sediment lithology. The following major geologic units were encountered at the Site, from the ground surface, or mudline, downward:

- **Soft Sediments.** The uppermost geologic unit consists of soft, dark brown, highly plastic sediments with varying proportions of clay, silt, and peat. Explorations indicate that this layer is 5 to 15 feet thick. Blow counts in this layer were generally 0 to 2 blows per foot. For cap-induced settlement evaluations, the majority of the settlement is expected to occur in this layer.
- **Shallow Alluvium.** This layer is characterized as a loose to medium dense sand with interbedded clay and silt, and has been interpreted to be a Shallow Alluvium layer. The thickness of the Shallow Alluvium appears to be greatest toward shore, approximately 10 to 20 feet, and thins offshore to approximately 5 feet thick. The Shallow alluvium is typically loose to medium dense with density increasing with depth. Significant amounts of organic sediments were generally not observed in this layer, but layers of silt encountered in this layer would be compressible in the event of cap placement.
- **Deeper Alluvium.** The Deeper Alluvium consists of medium dense to dense, coarse sand and gravel. For the purposes of cap stability, this layer is generally below the depth of interest. For cap-induced settlement evaluations, this layer is generally assumed to be incompressible.

Based on visual observations of the nearshore surface sediment, there is some coarse-grained material (silty sand) present along the shoreline. Although some of the explorations indicate relatively thick sand deposits in some of the nearshore areas, the coarse-grained material may not exist consistently along the shoreline or extend into the offshore area. For the purpose of this evaluation, the soft sediment layer was used for the analysis of the 1.5-foot thick sand cap—this is a conservative approach.

B4-3 Settlement Analyses

This section describes the preliminary analyses that were performed to estimate cap-induced primary consolidation settlement.

B4-3.1 Conceptual Cap Design Sections

The calculations presented herein were performed for two scenarios:

- **"No Prior Dredging":** 1.5-foot-thick sand cap placed directly over soft sediment; and
- **"With Prior Dredging":** 1.5-foot-thick sand cap placed after dredging of 1.5 feet of soft sediment.

An additional evaluation will be conducted for a third scenario: the Alternative 2 Amended reactive cap (no-prior dredging 4.5-feet-thick cap).

B4-3.2 Cap-Induced Load

The buoyant unit weight of the cap was assumed to be 70 pounds per cubic foot (pcf). For a 1.5-foot-thick cap, this assumption results in a stress increase of 105 pounds per square foot (psf) in the subsurface sediments and soils. For the scenario in which dredging is performed prior to cap placement, the overall stress increase is smaller and is based on the difference between the unit weight of the cap material and the unit weight of the sediment. For the dredging scenario, the stress increase was estimated to be 71 psf.

B4-3.3 Sediment Properties and Layer Thicknesses

The geotechnical properties of the sediments used in this analysis were based on the results of relevant RI sampling available to date, and laboratory and field testing data collected from the geotechnical reports by others. At this conceptual level of analysis, soil parameters, including compressibility and shear strength parameters, were largely estimated based on index properties and field observations in conjunction with engineering judgment. A single one-dimensional consolidation test (Shannon and Wilson 1997) on a sample of organic clay and silt was available for this analysis. The consolidation test results were used to estimate the compressibility parameters of the soft sediment. For the Shallow Alluvium, the compressibility parameters were estimated based on correlations with Atterberg limits. To assess the variability in settlement estimates for a particular geologic layer, a range of compressibility parameters was calculated based on the given range of Atterberg limits and consolidation test data.

Based on field investigations and subsequent laboratory testing conducted by others as part of early Site investigations, some of the geologic units are best characterized by a range of thicknesses and/or a range of physical properties. To assess the potential range of settlement resulting from these observed variations, three cases (termed "very high", "high", and "moderate" compressibility) were evaluated to reflect varying compressibility and geologic layer thickness. Each case used a unique set of input parameters and a settlement estimate was developed for each case. The intent of this evaluation is to bracket the potential range of settlement that may occur as a result of cap construction and to estimate the potential range of differential settlements that may occur given the heterogeneity at the Site.

The soil parameters that were assumed for the consolidation settlement analysis are provided in Table B4-1.

Table B4-1 – Compressibility Assumptions for Settlement Calculations

Analysis Layer	Parameter	Settlement Evaluation Scenarios		
		Lower-End Assumptions	Intermediate Assumptions	Higher-End Assumptions
1	Description	Soft Sediment	Soft Sediment	Soft Sediment
	Layer Thickness in ft	10	10	15
	Buoyant Unit Weight in pcf	22.6	22.6	22.6
	Overconsolidation Ratio (OCR)	1.3	1.3	1.3
	$C_r/(1+e_o)$	0.028	0.030	0.034
	$C_c/(1+e_o)$	0.35	0.40	0.45
2	Description	Shallow Alluvium	Shallow Alluvium	Shallow Alluvium
	Layer Thickness in ft	10	10	15
	Buoyant Unit Weight in pcf	42.6	42.6	42.6
	Overconsolidation Ratio (OCR)	1.3	1.3	1.3
	$C_r/(1+e_o)$	0.012	0.012	0.012
	$C_c/(1+e_o)$	0.15	0.15	0.15

Note:
Deeper Alluvium assumed to be incompressible for the purpose of this analysis.

B4-3.4 Settlement Magnitude

Spreadsheet calculations were performed to calculate primary consolidation settlement using the assumed subsurface profiles described in previous sections. The geologic units were divided into sub-layers. For each layer, settlement was calculated using the estimated modified compression index and stresses in the sediment and soils as described in many geotechnical engineering text books (e.g., Das 2010). The sediments and soils were assumed to be slightly overconsolidated-consolidated (overconsolidation ratio [OCR] = 1.3). Differential settlement may occur between areas "With Prior Dredging" and areas with "No Prior Dredging". Differential settlements were calculated as the difference in primary consolidation of "No Prior Dredging" and "With Prior Dredging". At the interface between these two areas, differential settlement is generally expected to be gradual, not abrupt. The edges of the dredge area can be sloped to create a more gradual transition between the two areas. The results of the settlement calculations are summarized in Table B4-2.

Table B4-2 – Estimated Cap-Induced Total and Differential Settlement

Scenario	Cap Thickness in Feet	Dredge Depth in Feet	Estimated Total Settlement from Primary Consolidation in Inches	Estimated Worst Case Differential Settlement in Inches[1]
Lower-End Estimates				
With Prior Dredging	1.5	1.5	4	8
No Prior Dredging	1.5	0	12	
Intermediate Estimates				
With Prior Dredging	1.5	1.5	5	9
No Prior Dredging	1.5	0	14	
Higher-End Estimates				
With Prior Dredging	1.5	1.5	6	10
No Prior Dredging	1.5	0	16	

Notes:
General – The assumptions for the settlement calculations are summarized in Table B4-1.
1. Differential settlements were calculated as the difference in primary consolidation of "No Prior Dredging" and "With Prior Dredging".

B4-4 Bearing Capacity

A traditional bearing capacity analysis was performed to estimate the maximum lift thickness that could be placed during construction.

B4-4.1 Method of Analysis

Appendix C of the Assessment and Remediation of Contaminated Sediments (ARCS) Program cap design guidance manual Guidance for In-Situ Subaqueous Capping of Contaminated Sediments (Palermo et al. 1998) describes a method of assessing stability of a cap placed on soft sediment. Refinements to this methodology are presented in a U.S. Army Engineer Research and Development Center Technical Note (Rollings 2000). The method is based on the bearing capacity theory applied to a shallow foundation on a subgrade, whereby the cap is considered a footing acting over a large area. In this case, the footing contact pressure is calculated as the submerged unit weight of the cap multiplied by its thickness:

$$q = \gamma' h \qquad \text{(EQ 1)}$$

Where:

q = "footing" contact pressure in psf

γ' = submerged unit weight of cap in pcf

h = cap lift thickness in ft

Due to the soft nature of the sediments to be capped, the undrained soil shear strength is appropriate. After placement of the initial cap lift, the pore pressures will dissipate as part of the consolidation process and the shear strength of the underlying sediment will improve. The ultimate bearing capacity is calculated as follows:

$$q_{ult} = s_u N_c \qquad \text{(EQ 2)}$$

Where:

q_{ult} = ultimate bearing capacity in psf

s_u = undrained shear strength of sediment in psf

N_c = bearing capacity factor (N_c = 5.7 for undrained conditions ($\phi = 0$))

The allowable bearing capacity (q_{allow}) is calculated as follows:

$$q_{allow} = q_{ult} / FS \qquad \text{(EQ 3)}$$

Where:

FS = factor of safety for bearing capacity under short-term conditions (FS = 1.5 was used)

By combining equations EQ 1, EQ 2, and EQ 3, the maximum lift thickness is calculated as follows:

$$h_{max} = (s_u N_c) / (FS\ \gamma')$$

B4-4.2 Assumptions

For this preliminary bearing capacity assessment, relatively conservative assumptions were made in terms of the undrained shear strength of the sediments to be capped. It was assumed that the sediments to be capped are very soft. The following average undrained strengths were assumed:

- For "No Prior Dredging": s_u = 15 psf;
- For "With Prior Dredging": s_u = 25 psf.

The cap was estimated to have a submerged unit weight of 70 pcf.

B4-4.3 Bearing Capacity Assessment Results and Conclusions

For this preliminary assessment, the following maximum lift thicknesses were calculated:

- For “No Prior Dredging”: h_{max} = 9 inches
- For “With Prior Dredging”: h_{max} = 16 inches

These results are based on relatively conservative assumptions in terms of the undrained strength of the underlying sediment. There are no existing strength data for the sediments; therefore, the estimates of bearing capacity have significant uncertainty. Prior to design, design-level geotechnical data should be collected to refine the analysis. Should the shear strength of the underlying sediment actually be as low as assumed for this assessment, the cap will need to be placed in two lifts. The thicknesses provided above are the maximum lift thicknesses for the initial lift. Following placement of the initial lift thickness, the underlying sediment will need to be allowed to consolidate and gain strength before additional cap material is placed. The time between placement of the initial lift and second lift will be estimated during design based on design-level data. If the sediment is stronger than estimated herein, it may be possible to place the cap in one lift. Generally, the cap will need to be built up gradually to the maximum lift thickness before construction is stopped to allow consolidation to occur. If the sediment is very soft, it may be advisable to first place a geotextile fabric to provide additional support.

B4-5 Seismic Considerations

The seismic hazard at the Site, particularly in the upland setting, has been analyzed and discussed by others (Aspect 2009). The conclusions of the upland studies are based on current building codes. Building codes are generally not directly applicable to earthen structures. No guidance currently exists for seismic considerations for environmental cleanup projects and sediment capping projects in particular. However, for some Comprehensive Environmental Response, Compensation, and Liability Act (CERLCA) projects, a design seismic event with a 10% probability of exceedance in 50 years (475-year return period) has been used. This level of event seems appropriate and originated from port facility design where it was referred to as the Contingency Level Event (CLE). Per the 2008 U.S. Geological Survey seismic hazard maps (Kramer 2008), the peak ground acceleration for rock outcrop associated with the 475-year event is 0.3 g (g = acceleration of gravity). Some amplification is to be expected due to the soft soils at the Site. Under this event, some liquefaction of the sand cap and some of the underlying soils is possible. The consequences of seismic shaking will need to be evaluated during design. Generally, the in-water slopes to be capped are fairly gentle (approximately 10H:1V). Seismic stability of an *in situ* sediment cap was assessed for the Palos Verdes Shelf off the coast of Los Angeles, California (USACE 1999). For the Palos Verdes site, it was concluded that a sand cap would be reasonably stable on slopes of 5 degrees or less; this is generally similar to the conditions at the Site. Analyses to be performed during design may indicate that some form of stabilization will be required. Stabilization may consist of a terraced configuration with “rock ribs” between sediment cap terraces. The rock ribs would reduce lateral movement of the cap and reduce the need for repairs after a significant seismic event. Some settlement may also occur as a result of seismic liquefaction. Generally, sediment caps should be inspected after significant seismic events and repairs performed as necessary.

B4-6 Considerations for Amended Sand Cap

Alternative 2 includes a 4.5-foot-thick amended sand reactive cap that would be placed in dredge area DA-6. This cap consists of the following layers (from top to bottom):

- 0.5 feet of aquatic habitat friendly material
- 2 feet of clean sand
- 2 feet of sand (90%) and organoclay (10%) mix

The individual layer and overall thicknesses are nominal for FS purposes. The final thicknesses would be defined during design.

The amended sand cap covers a nearshore area that is approximately 240 feet long by 140 feet wide. Based on existing subsurface exploration data presented in the Remedial Investigation report (Appendix E; Anchor QEA and Aspect 2012), the area closest to the shoreline is underlain predominantly by sand. The assumption that sandy subsurface conditions exist under the amended sand cap is different from the subsurface conditions assumed for the 1.5-foot sand cap provided in Section B4-3.3. Explorations advanced outside of dredge area DA-6 indicate the existence of soft sediments that would likely settle significantly under the weight of the cap. Thus, the assumptions for the 1.5-foot cap may be valid further offshore.

For the 4.5-foot cap, the sand along the shoreline is expected to provide sufficient bearing capacity and will not compress significantly. The transition from sandy subsurface conditions to softer conditions will need to be delineated further during design based on additional subsurface explorations. The 4.5-foot cap will need to be properly engineered during design to account for the actual subsurface conditions. If the 4.5-foot cap is to be placed on soft sediments, it may be necessary to use high-strength geotextile to improve bearing capacity. Settlement may also occur over time and the 4.5-foot cap thickness may need to be replenished over time. However, in general, cap material placed on sand in the area along the shoreline is not expected to settle significantly. Therefore, the creation of shallow-water habitat in these areas is anticipated to be feasible and not expected to be affected by settlement.

B4-7 Conclusions

A series of geotechnical evaluations were performed to assess the constructability and stability of caps that may be constructed at the Site. Evaluations were also performed to estimate the amount of primary consolidation settlement that may be expected following placement of a subaqueous cap. Based on these evaluations, a subaqueous cap is generally considered feasible under the conditions that were evaluated herein. Caps constructed over soft sediments generally need to be placed in thin lifts; this will require the use of special construction techniques (e.g., the use of a spreader box). For cap design, it will be necessary to collect additional geotechnical data to better characterize the sediments and soils in the capping areas, in terms of shear strength, stress history, and compressibility. Additional geotechnical design analyses will need to be performed, particularly to assess the seismic stability of the cap. It may be necessary to install stabilizing measures such as rock ribs to improve seismic performance. Lastly, it should be noted that caps generally need to be monitored to assess their performance. If deficiencies are discovered during monitoring events, repairs may be needed. An inspection should be performed following a significant seismic event and repairs performed as necessary. Costs associated with monitoring and repairs need to be included in cost estimates, and funds for monitoring and repairs set aside if capping is selected.

Additionally, some alternatives include thinner physical isolation caps (e.g., 6 inches of sand) and a reactive cap consisting of an organoclay reactive core mat (RCM) overlain by approximately 6 inches of sand cap. Although, these caps were not specifically addressed in the evaluations above, settlement is expected to be less than the calculated settlement estimates presented above; therefore, they are generally considered feasible. RCMs also typically include the use of geosynthetic materials that can improve cap performance in terms of stability and differential settlement. Geosynthetic materials such as geotextiles may be added to sand caps to improve stability, provide separation between contaminated sediment and the cap, and provide demarcation to allow easier cap monitoring.

B4-8 References

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APPENDIX B5

Sheet Pile Enclosure Calculations

Contents

List of Tables

B5-1 Introduction and Background

Several of the remedial alternatives presented in this Feasibility Study (FS) for the Quendall Terminals Site (Site) include the use of a temporary sheet pile enclosure. As part of this FS, preliminary analyses were performed to select appropriate sheet pile sections and lengths for the various alternatives.

Dredging of nearshore sediments to various depths is included in 7 of the 10 alternatives presented in the FS. The various wall alignments are shown on the figures in the main text. For each of these 7 alternatives, a temporary sheet pile wall would isolate the nearshore dredge area from the open water of Lake Washington. Dredging within the enclosure would be performed with barge-mounted equipment and potentially land-based equipment along the shoreline where there may not be adequate draft for a barge.

B5-2 General Conditions

B5-2.1 Lake Water Levels

Lake Washington water levels are controlled by the Ship Canal Locks and do not vary significantly, generally only by 2 feet over the year. The lake is raised up to a targeted high water elevation of 18.67 feet NAVD88 in the summer months and low water elevation of 16.67 feet NAVD88 in the winter. A water level of elevation 18.67 feet NAVD88 was assumed for analysis purposes.

B5-2.2 Generalized Subsurface Conditions

Subsurface conditions used for this analysis were based in part on a review of existing geotechnical engineering reports for the Site (Aspect 2009). Geotechnical borings logs and sediment core logs collected as part of the Remedial Investigation (RI) (Anchor QEA and Aspect 2012), as well as laboratory data, and historical geotechnical borings by others (Twelker and Associates 1973, Shannon and Wilson 1997) were also used in assessing subsurface conditions and properties.

The following major geologic units were encountered at the Site, from the ground surface, or mudline, downward:

- **Soft Sediments.** The uppermost geologic unit consists of soft, dark brown, highly plastic sediments with varying proportions of clay, silt, and peat. Explorations indicate that this layer is 5 to 15 feet thick. Blow counts in this layer were generally 0 to 2 blows per foot.

- **Shallow Alluvium.** This layer is characterized as a loose to medium dense sand with interbedded clay and silt, and has been interpreted to be a Shallow Alluvium layer. The thickness of the Shallow Alluvium appears to be greatest toward shore, approximately 10 to 20 feet, and thins offshore to approximately 5 feet thick. The Shallow Alluvium is typically loose to medium dense with density increasing with depth. Significant amounts of organic sediments were generally not observed in this layer.

- **Deeper Alluvium.** The Deeper Alluvium consists of medium dense to dense, coarse sand and gravel. For the purposes of cap stability, this layer is generally below the depth of interest.

Based on visual observations of the nearshore surface sediment, there is some coarse-grained material (silty sand) present along the shoreline. However, the coarse-grained material may not extend beyond the surface or into the offshore area. For the purpose of this evaluation the soft sediment layer was used for the analysis—this is a conservative approach.

B5-3 Methodologies

B5-3.1 Method of Analysis

The public domain computer program ProSheet (developed by Arbed) was used to perform the sheet pile wall analyses. ProSheet uses the Blum theory to calculate embedment depths, wall deflections, forces, and bending moments.

B5-3.2 Earth Pressure Calculations

Active and passive earth pressures were used for the geotechnical design of the enclosure walls. Earth pressures were calculated using Coulomb earth pressure theory (ASCE 1996).

Earth pressures for drained (long-term loading) analyses were calculated by multiplying the effective vertical stress of the soil by the appropriate earth pressure coefficient. Earth pressure coefficients were calculated using Coulomb earth pressure theory for active and passive pressures. For drained analyses, the soil's angle of internal friction and an appropriate wall friction angle were used to calculate the earth pressure coefficients. Soil parameters used for design are provided in subsequent sections of this memorandum.

Earth pressures for undrained (short-term loading) analyses were calculated as follows:

Active: $\sigma_a = \sigma'_v - 2s_u$

Passive: $\sigma_p = \sigma'_v + 2s_u$

Where:

σ_a = active lateral earth pressure

σ'_v = effective vertical stress

s_u = undrained shear strength

σ_p = passive lateral earth pressure

Using the above equation for calculation of the active earth pressure, the active pressure could become negative at low effective vertical stresses. Where the calculated active pressure was negative, the active pressure was assumed to be equal to zero. Undrained shear strength and unit weights that were used for the soils are provided in subsequent sections of this memorandum.

B5-3.3 Calculation of Design Soil Shear Strength for Passive Earth Pressures

Wall stability calculations were performed using both drained and undrained analyses. Soil parameters assumed for the analyses are provided later in this appendix. For calculation of embedment depths required for wall stability, factors of safety were applied to the soil strength used for calculation of passive earth pressures. No factors of safety were applied to active earth pressures.

Design shear strength parameters used for calculation of passive earth pressures were calculated as follows:

- Undrained Strength: $s_{u,design} = s_u / FS_p$
- Drained Strength: $\tan(\varphi_{design}) = \tan(\varphi) / FS_p$

Where:

s_u = undrained shear strength

ϕ = angle of internal friction (drained strength parameter)

FS_p = factor of safety applied to soil strength prior to calculation of passive earth pressures

B5-3.4 Factors of Safety

Using guidelines provided in the Design of Sheet Pile Walls (ASCE 1996), factors of safety for calculation of wall embedment depths were selected based on the loading case, type of loading, and type of soil. The walls were designed using usual, unusual, and extreme loading cases per USACE design procedures (ASCE 1996). These loading cases correlate with the likeliness for the load to occur. More severe and less likely loading cases are generally assigned smaller factors of safety than less severe loading cases that occur regularly under normal operating conditions. Table B5-1 lists the factors of safety used for passive earth pressure calculations.

Table B5-1 – Factors of Safety

Loading Case	FS_p
Usual	1.5
Unusual	1.25
Extreme	1.1

Note: FS_p = factor of safety applied to soil strength prior to calculation of passive earth pressures

B5-3.5 Forces and Moments for Structural Design

To avoid compounding of factors of safety, the structural components were designed using a factor of safety of 1 on the soil side to calculate the forces and moments. To calculate required embedment depths, the analyses were then repeated applying the appropriate factor of safety on the passive earth pressure side for each of the loading conditions (i.e., usual, unusual, and extreme loading conditions). Allowable stresses for structural design were calculated taking into account the various loading conditions, as described in the following sections.

B5-3.6 Allowable Stresses for Steel Sheet Piling

Allowable stresses for steel for usual loading conditions were calculated per the U.S. Army Corps of Engineers (USACE) design procedures (ASCE 1996) as follows:

$f_b = 0.5\ f_y$ (combined bending and axial load)

$f_v = 0.33\ f_y$ (shear)

For the unusual loading conditions, the allowable stress equations were increased 33 percent above that for usual loading conditions:

$f_b = 1.33\ (0.5\ fy) = 0.67\ fy$

For the extreme loading conditions, the allowable stress equations were increased 75 percent above that for usual loading conditions:

$f_b = 1.75\ (0.5\ f_y) = 0.875\ f_y$

Where:

f_b = combined bending and axial load

f_v = shear stress

f_y = yield stress of the steel

The increases in allowable stress are appropriate given the infrequent, short-term loading conditions on structural elements that can be subjected to greater load.

B5-3.7 Wall Deflection Limitations

Sheet pile sections were selected based on both bending moments and deflections. Deflections were calculated for conditions with and without wave loads. Wave loads are transient loads that only occur for brief moments. The dynamic nature of these loads cannot be modeled in any available sheet pile analysis software. The wave loads were modeled as static loads and it is assumed that this results in overestimation of the deflections that include wave loads. As part of the selection of the sheet pile sections, top-of-wall deflections were limited as follows:

- Maximum deflection for deflection calculations without wave load: 5 inches
- Maximum deflection for deflection calculations with wave load: 10 inches

B5-4 Assumptions

B5-4.1 Top-of-Wall Elevation

A top-of-wall elevation of 23 feet NAVD88 was selected. This elevation was selected such that overtopping would not occur from high water levels and most wave actions.

B5-4.2 Design Sections

Due to the similarities in enclosure alignments and dredge depths, alternatives were grouped for analysis as follows:

- Group 1: Alternatives 4, 5, and 6
- Group 2: Alternatives 7 and 8
- Group 3: Alternatives 9 and 10

Differences between the Groups are summarized as follows:

- **Group 1:** Alternatives 4, 5, and 6 are characterized by a relatively short sheet pile alignment with a length of approximately 700 feet. The mudline elevation on the lakeside wall does not vary significantly from the lowest elevation of approximately 6 feet to approximately 8 feet NAVD88. The generalized design section was based on the outer lakeside wall due to wave loads, largest water depth, and overall most severe loading conditions. A conservative dredge depth of 8 feet of excavation was analyzed.
- **Group 2:** Alternatives 7 and 8 include a longer sheet pile alignment with a length of approximately 1,260 feet. The mudline elevation of the longest bay side wall varies from the lowest elevation of approximately 3 feet to approximately 10.5 feet NAVD88. The wall was analyzed at the northeast portion due to the deepest water depth in this region, influence from wave loads, and deepest excavation near the wall. A dredge depth of 11.5 feet of excavation was analyzed.
- **Group 3:** Alternatives 9 and 10 include the longest sheet pile alignment with a length of approximately 1,530 feet and the deepest excavation with material being removed down to the Shallow Alluvium layer. The mudline elevation varies significantly across the alignment with elevations of approximately 0.5 feet to 11 feet NAVD88. Due to the much larger excavation depths, two wall sections were analyzed, one on the lake side wall and another for the return wall towards the shoreline with excavation depths of 24 feet and 28 feet, respectively.

Table B5-2 shows the design sections used for the preliminary analyses.

Table B5-2 – Design Sections

Description	Group 1	Group 2	Group 3 Section A	Group 3 Section B
Sediment Surface Elevation	6	3	11	8
Thickness of Soft Sediment (feet)	5	5	7	5
Thickness of Shallow Alluvium[1] (feet)	14	14	20	19

Note:
[1] Shallow Alluvium is underlain by Deep Alluvium.

B5-4.3 Soil Parameters

Soil parameters were based on available subsurface information. Shear strength parameters were selected based on correlations with Standard Penetration Test (SPT) blow counts and soil plasticity data, in conjunction with engineering judgment. Table B5-3 shows the soil parameters used for this feasibility-level assessment.

Table B5-3 – Soil Parameters

Soil Parameter	Soft Sediment	Shallow Alluvium	Deeper Alluvium
Total Unit Weight, γ_T (pcf)	85	105	125
Submerged Unit Weight, γ' (pcf)	22.6	42.6	62.6
Angle of Internal Friction, ϕ' (degrees)	15	20	36
Wall Interface Friction Angle, δ (degrees)	7	10	18
Undrained Strength, S_u (psf)	75	500	NA

Notes:
NA = not applicable
pcf = pounds per cubic foot
psf = pounds per square foot

B5-4.4 Design Loads

For this feasibility-level assessment, design loads consisted of earth pressures, hydrostatic loads due to water level differentials, and wave action. The calculation of earth pressures is discussed above. Assumptions regarding hydrostatic loads and wave loading are discussed below.

Hydrostatic Loads

Some water level changes may occur during dredging on the outside and inside of the enclosure. Generally, the water level within the enclosure would need to be controlled by the contractor to keep water level differentials and associated hydrostatic loads on the wall relatively small. For analysis purposes, the water level inside the enclosure was assumed to be 1 foot below the lake level. The analyses were performed for the summer lake conditions with an elevation of 18.67 feet NAVD88 as this would result in the greatest hydrostatic load on the wall.

Wave Loads

Wave loads were taken into account for the various scenarios. Both wind-induced waves and vessel-induced waves were analyzed in Appendix B3 – Cap Armor Layer Evaluation It was determined that for the majority of the wall, non-breaking waves needed to be taken into account as the depths along the longer bay side portions of the enclosure are sufficient to be above the transitions zone to breaking waves. The occurrence of direct breaking waves against the enclosure is unlikely and forces resulting from such impacts would only last for short durations (on the order of hundredths of a second). Wall stability analyses were analyzed for 1-, 2.5-, and 3.5-foot wave heights for usual, unusual, and extreme loading conditions, respectively. Wave forces were calculated using the Shore Protection Manual (USACE 1984). The calculations are presented on Table B5-4.

B5-4.5 Steel Grade

The selected steel grade is ASTM A572 – Grade 50.

B5-5 Analysis Results

The load combinations and results for the feasibility-level analyses are provided in Table B5-5 (attached). Table B5-6 shows the sheet pile lengths and sections that would be required based on those results.

Table B5-6 – Sheet Pile Length and Section

Description	Sheet Pile Length (feet)	Sheet Pile Section[1]
Group 1: Alternatives 4, 5, & 6	47	AZ17-700
Group 2: Alternatives 7 & 8	50	AZ24-700
Group 3: Alternatives 9 & 10		
North wall	60	AZ50
Bay side wall	60	AZ50

Note:
[1]Section designations presented in this table are for sections made by ArcelorMittal (available through Skyline Steel). Similar sections with similar properties are also available through other suppliers.

B5-6 Conclusions

Based on the feasibility-level analyses presented herein, a sheet pile enclosure would be a generally feasible technology to accommodate dredging of the nearshore sediments. The results presented herein are preliminary in nature. Additional analyses would be required during design to refine the selection of the sheet piles.

B5-7 References

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Table B5-4 of Appendix B5 - Wave Force Calculations for Enclosure Wall

Quendall Terminals
Renton, Washington

Assumptions / Input Parameters for Figures 7-90 through 7-92 (USACE 1984):

χ = 1.0 wave reflection coefficient

γ_w = 62.4 pcf unit weight of water

Water Level	Water Level Elevation (ft)	Mudline Elevation (ft)	d (ft)	H_i (ft)	T (s)	H_i/d [a]	$H_i/(gT^2)$ [b]	h_0/H_i [c]	y_c [d]	$(y_c\text{-}d)$	$F/(\gamma_w d^2)$ [e]	F [f] (lbs/ft)	h_F (ft)	Load Application Elevation (ft NGVD29)
Group 1: Alternatives 4, 5, & 6														
Usual Event	18.67	6	12.67	1	2	0.08	0.0078	0.15	13.82	1.15	0.02	200	8.9	14.9
Unusual Event	18.67	6	12.67	2.5	3	0.20	0.0086	0.200	15.67	3.00	0.09	902	8.9	14.9
Extreme Event	18.67	6	12.67	3.5	3.5	0.28	0.0089	0.255	17.06	4.39	0.16	1603	8.9	14.9
Group 2: Alternatives 7 & 8														
Usual Event	18.67	3	15.67	1	2	0.06	0.0078	0.15	16.82	1.15	0.02	306	11.0	14.0
Unusual Event	18.67	3	15.67	2.5	3	0.16	0.0086	0.195	18.66	2.99	0.05	766	11.0	14.0
Extreme Event	18.67	3	15.67	3.5	3.5	0.22	0.0089	0.215	19.92	4.25	0.1	1532	11.0	14.0
Group 3: Alternatives 9 & 10														
North wall														
Usual Event	18.67	11	7.67	1	2	0.13	0.0078	0.17	8.84	1.17	0.04	147	5.4	16.4
Unusual Event	18.67	11	7.67	2.5	3	0.33	0.0086	0.300	10.92	3.25	0.24	881	5.4	16.4
Extreme Event	18.67	11	7.67	3.5	3.5	0.46	0.0089	0.435	12.69	5.02	0.43	1578	5.4	16.4
Bay side wall														
Usual Event	18.67	8	10.67	1	2	0.09	0.0078	0.15	11.82	1.15	0.02	142	7.5	15.5
Unusual Event	18.67	8	10.67	2.5	3	0.23	0.0086	0.220	13.72	3.05	0.11	781	7.5	15.5
Extreme Event	18.67	8	10.67	3.5	3.5	0.33	0.0089	0.300	15.22	4.55	0.23	1634	7.5	15.5

Note:

Calculations are based on methods provided in the 4th edition of the Shore Protection Manual (U.S. Army Corps of Engineers (USACE) 1984).

Footnotes:

a. $H_i/d > 0.67$ --> Wave is likely a breaking wave.

b. Obtained values from Figure 7-92 (USACE 1984).

c. Obtained values from Figure 7-90 (USACE 1984). For $H_i/d < 0.10$, values obtained from 0.10 curve.

d. Value based on Equation 7-73 (USACE 1984).

e. Obtained values from Figure 7-91 (USACE 1984).

f. Hydrostatic force not included.

Acronyms and Abbreviations:

d = water depth
F = wave force (includes hydrostatic component)
ft = feet
g = acceleration of gravity (32.2 ft/s^2)
h_F = distance between mudline and force application point
H_i = wave height
h_0 = height of clapotis orbit above still water level
lbs = pounds
NGVD29 = National Geodetic Vertical Datum of 1929
pcf = pounds per cubic foot
s = seconds
T = wave period
y_c = distance between mudline and wave crest
γ_w = unit weight of water

Table B5-5 of Appendix B5 - Load Combinations and Analysis Results for Enclosure Wall

Quendall Terminals
Renton, Washington

Load Combinations				Results for Drained Analyses				Results for Undrained Analyses			
Description	Dredge Depth (ft)	Water Level Difference (ft)	Wave Height (ft)	Required Minimum Sheet Pile Length (ft)	Required Minimum Section Modulus (in^3/ft)[1)]	Deflection with wave load (in)[2)]	Deflection without wave load (in)[2)]	Required Minimum Sheet Pile Length (ft)	Required Minimum Section Modulus (in^3/ft)[1)]	Deflection with wave load (in)[2)]	Deflection without wave load (in)[2)]
Group 1: Alternatives 4, 5, & 6				*Selected Section AZ17-700* [3)]				*Selected Section AZ17-700* [3)]			
Usual Event	8	1	1	42.3	16.14	3.4	2.8	33.1	9.51	1.2	0.9
Unusual Event	8	1	2.5	46.7	18.71	5.8	2.8	34.2	11.78	2.2	0.9
Extreme Event	8	1	3.5	42.6	19.41	8.3	2.8	35.0	12.51	3.5	0.9
Group 2: Alternatives 7 & 8				*Selected Section AZ24-700* [3)]				*Selected Section AZ24-700* [3)]			
Usual Event	11.5	1	1	49.3	28.71	4.7	3.8	42.0	16.55	2.1	1.5
Unusual Event	11.5	1	2.5	48.7	26.64	6.2	3.8	42.1	16.41	3.0	1.5
Extreme Event	11.5	1	3.5	48.5	26.78	8.9	3.8	42.9	17.61	4.8	1.5
Group 3: Alternatives 9 & 10				*Selected Section AZ50* [3)]				*Selected Section AZ50* [3)]			
North wall											
Usual Event	28	1	1	59.6	62.03	6.2	5.8	53.6	25.61	2.4	2.1
Unusual Event	28	1	2.5	58.1	56.82	8.2	5.8	53.5	28.86	4.0	2.1
Extreme Event	28	1	3.5	57.0	50.67	10.3	5.8	53.3	29.01	5.7	2.1
Bay side wall											
Usual Event	24	1	1	58.4	52.90	5.1	4.7	53.4	25.53	2.4	2.1
Unusual Event	24	1	2.5	57.2	50.43	7.1	4.7	53.4	28.80	4.0	2.1
Extreme Event	24	1	3.5	56.2	45.40	9.0	4.7	53.2	28.94	5.7	2.1

Notes:

1) Based on bending moments.

2) Used both bending moments and deflections for selection of section. Deflections calculated with selected section properties for two scenarios: with calculated wave force from Table B5-2 and without wave force.

3) Assumed steel grade is ASTM A572 - Grade 50.

Acronyms and Abbreviations:

ft = feet
in = inches

APPENDIX C

Description of Technologies and Process Options for DNAPL, Soil, Groundwater, and Sediment

Table of Contents

C1 Introduction

The information in this appendix provides additional detail on remedial technologies and process options presented in Section 5 of the Feasibility Study (FS) to address dense non-aqueous phase liquid (DNAPL), soil, groundwater, and sediment.

C2 DNAPL Technologies and Process Options

C2.1 DNAPL Institutional Controls

Potentially applicable institutional controls for DNAPL include the following:

- Fences and warning signs to control access to the Quendall Site (Site) or to specific areas of the Site such as the nearshore area in the vicinity of Quendall Pond.
- Deed restrictions, such as restricting land use, construction, and soil excavation without U.S. Environmental Protection Agency (EPA) approval.
- Use restrictions and monitoring requirements to prevent disturbance of caps or other engineered controls.

Each of the above institutional controls is potentially effective at preventing exposure to hazardous substances, is easy to implement, and can be implemented at relatively low cost. Institutional controls have commonly been implemented as part of a remedy at similar sites. Therefore, they have been retained as representative institutional control process options.

C2.2 DNAPL *In Situ* Containment

The lateral mobility of DNAPL can be controlled by installing impermeable vertical barriers across potential DNAPL flow paths. At the Site, vertical barriers can be keyed into low-permeability soil layers in the Shallow Alluvium to limit horizontal liquid-phase migration. Vertical barriers would not prevent vertical DNAPL migration through discontinuities in low-permeability soil layers.

Free-phase DNAPL is typically present at the Site in relatively thin layers; DNAPL mobility at the Site is already limited by low-permeability soils or sediments (see Section 3.5 of the FS). However, this technology could offer additional protection by limiting migration of free-phase DNAPL. Note that placing an impermeable vertical barrier may also require collecting and treating groundwater (discussed below in Section 4) to prevent spreading of the contaminated groundwater plume as well as downgradient monitoring wells to confirm that DNAPL is being retained behind the vertical barrier. Impermeable barriers to prevent DNAPL migration are considered applicable only to upland Site areas.

Process options for impermeable vertical barriers include the following:

- **Slurry Walls.** Can be constructed using a one-pass continuous trencher or by traditional trench excavation and backfilling.
  - **High-density polyethylene (HDPE) or soil-bentonite slurry walls constructed using a one-pass continuous trencher.** Shallow subsurface debris (pipes, rubble) may need to be cleared with an excavator prior to using the trencher. Maximum depth of trenching using this method is approximately 35 feet (DeWind 2010). Unit costs for this option are typically around $6/vertical square foot (VSF) for slurry walls and $20/VSF for HDPE walls (Banks et al. 2006).
  - **Slurry walls constructed by excavating a trench and backfilling with a bentonite, cement-bentonite, or soil-bentonite slurry.** Slurry walls can also be constructed by driving vertical plates and injecting grout as the plate is removed. Unit costs for this option typically range from $2 to $10/VSF (Navy Website 2010) for walls up to 80 feet deep.
- **Sheet Pile Wall.** Interlocking sheet pile sections constructed of steel or HDPE. Sheets are either driven or vibrated into the ground, and joints are sealed with grout to prevent leaking. Unit costs for this option typically range from $25 to $80/VSF (Navy Website 2010).
- **Grout Curtain.** Slurry walls constructed by injection of cement or bentonite grout into soil (jet grouting) to construct a grout curtain. This technology can be used to construct very deep barriers, although establishing a continuous wall of consistent thickness is more difficult, and the resulting permeability is often higher than walls constructed by other methods. Unit costs for this option range from $40 to $200/VSF (Navy Website 2010) for walls up to 400 feet deep.

The process options discussed above potentially are implementable at the Site and effective for DNAPL. Sheet pile and grout curtain walls are significantly more costly than slurry walls, and the greater depths obtainable with a grout curtain are not necessary for the Site because DNAPL is present at a maximum depth of 34 feet below ground surface. Both methods of slurry wall installation (trench excavation and one-pass continuous trencher) have similar costs, but the trench excavation method has been more conventionally used and would be able to more easily cope with subsurface debris, which is expected to be present in some Site locations. Therefore, slurry wall installation via trench excavation is retained as a representative process option for impermeable vertical barriers.

C2.3 DNAPL *In Situ* Treatment

Potentially applicable *in situ* DNAPL treatment technologies include *in situ* thermal treatment (low-temperature thermal treatment, mid-temperature thermal treatment, and high-temperature thermal treatment), chemical oxidation, and *in situ* stabilization. Each of these technologies is discussed below.

C2.3.1 In Situ Thermal Treatment

Subsurface heating can be used to destroy or volatilize organic chemicals present in soil, sediment, and groundwater. This technology typically includes a network of heating or injection wells to heat the subsurface, and a network of extraction wells to remove contaminated soil vapor, groundwater, and DNAPL from the subsurface. Contaminated fluids are treated above ground, typically by a combination of physical separation (to remove DNAPL), adsorption (to remove dissolved contaminants), and thermal oxidation (to destroy contaminated vapors).

Process options for *in situ* thermal treatment include the following:

- **Hot Water Injection.** Hot water is injected into the subsurface, decreasing DNAPL viscosity and raising the solubility of organic compounds.
- Steam Injection. Steam is injected into the subsurface, volatilizing or destroying (by pyrolysis) organic compounds. This heating method is considered the most cost-effective method of heat transfer to permeable soils, but effectiveness is limited in low-permeability soils.
- **Electrical Resistance Heating (ERH).** A voltage is applied to subsurface electrodes installed in vertical boreholes. The electrical resistivity of site soils creates heat. The efficiency of this method depends on the subsurface electrical properties, including soil type and moisture content.
- **Thermal Conduction Heating (TCH).** Vertical wells are heated, typically using in-ground electrical heaters, and the heat is transferred to subsurface soils via the soil's thermal conductivity. This method of heating provides relatively consistent heating regardless of soil type.

These heating methods are developing technologies and may require bench, pilot, or treatability testing prior to design and implementation.

Thermal treatment methods are considered applicable only to Site upland areas. Thermal treatment of Lake Washington sediments would be highly inefficient because of heat loss to the lake and would mobilize contaminants[1] that could not be reliably captured, resulting in aquatic habitat degradation.

Operating temperatures can be varied depending on remedial action objectives (RAOs), with three general technology types based on the level of heating:

- **Low-Temperature Heating.** The subsurface is heated to a temperature below the boiling point of water.
- **Mid-Temperature Heating.** The subsurface is heated to the boiling point of water.
- **High-Temperature Heating.** Also called *in situ* thermal desorption, the subsurface is heated above the boiling point of water.

Each of these technologies is discussed below.

Low-Temperature Heating. Heating the subsurface to temperature less than the boiling point of water would reduce the DNAPL viscosity and increase the solubility of DNAPL constituents for enhanced physical recovery. It would also volatilize the most volatile compounds. A portion of residual DNAPL would remain coated to soil after treatment.

Low-temperature heating is a developing technology for treatment of creosote and coal tar and has been used to enhance physical recovery of non-aqueous phase liquid (NAPL). The only full-scale applications of low-temperature heating to creosote or coal tar sites

[1] Mobilizing contaminants including decreasing the density of creosote and coal tar to below the density of water, creating a floating product (LNAPL).

have been applied as the first phase of a higher temperature heating method. Low temperature heating has most often been applied to sites containing chlorinated solvents where water-solvent mixtures have azeotrope boiling points less than 100 degrees C (EPA 2004). Based on case studies for these sites, unit costs are expected to range between $60 and $250/cy depending on the size of the area treated.

Process options to achieve low-temperature heating include hot water injection, ERH, and TCH. These technologies are likely applicable to the Site, although the resistivity of Site soils would have to be tested to verify the effectiveness of ERH. Hot water injection would have limited effectiveness based on the prevalence of low-permeability soil layers in the Shallow Alluvium where DNAPL is located. Because Site soils are heterogeneous, the technology would require substantial groundwater controls, such as barrier walls and/or a DNAPL recovery system, to prevent contaminant mobilization to Lake Washington. Site subsurface conditions[2] would require a relatively dense network of extraction and heating wells.

This technology has had limited full-scale application and is not likely to be cost-effective when compared to other technologies for addressing DNAPL for the following reasons:

- Mid-temperature heating (described below), is slightly more expensive but would remove much more contaminant mass.
- The cost of low-temperature heating is comparable or higher than for *in situ* stabilization, which would be more effective in addressing both free-phase and residual DNAPL in heterogeneous soil conditions.

Therefore, this technology has not been carried forward for remedial alternative assembly.

Mid-Temperature Heating. Heating the subsurface to the boiling point of water would improve contaminant removal, when compared to low-temperature heating, by further reducing the DNAPL viscosity and increasing contaminant solubility. Many of the Site chemicals of concern (COCs), including benzene and naphthalene, would be volatilized and removed, but a significant fraction of semivolatile compounds, such as carcinogenic polycyclic aromatic hydrocarbons (cPAHs), would remain in soil. Residual material treated by this technology would be relatively immobile and contain compounds of lower solubility, significantly reducing the amount of contaminant leaching (Baker and Herron 2010).

Mid-temperature heating is a developing technology for treatment of creosote and coal tar. Full-scale applications of mid-temperature heating to creosote or coal tar sites include the Visalia Pole Site in California, where creosote-containing soil was treated using steam. The unit cost of treatment at Visalia was approximately $100/cy (USACE 2009) to treat 115,000 cy of soil.

[2] Including a high water table; contamination distributed over a broad, shallow area; presence of high-organic soils such as peat and organic silt, which reduces removal efficiency of contaminants; and the presence of highly heterogeneous soils.

Similar to low-temperature heating, mid-temperature heating has most often been applied to sites containing chlorinated solvents; based on case studies for these sites, unit costs are expected to range between $100 and $450/cy depending on the size of the area treated and subsurface conditions (NAVFAC 2007). As previously stated, Site subsurface conditions would require a relatively dense network of extraction and heating wells for this option.

Process options to achieve mid-temperature heating include steam, ERH, and TCH. These technologies may be applicable to different portions of the Site, although the use of steam in the Shallow Alluvium may be inefficient based on the presence of low-permeability silts and peat. In some cases, a combination of steam, ERH, and TCH may be used to realize the benefits of each technology. Mid-temperature heating was not retained in the Draft Evaluation of Groundwater Restoration Potential Technical Memorandum (Aspect and Anchor QEA 2011) since it is unlikely to achieve MCLs.

High-Temperature Heating. In high-temperature heating, also called *in situ* thermal desorption, most DNAPL constituents, including semivolatile compounds, such as cPAHs, would be removed or destroyed *in situ*. Variation in the degree of contaminant reduction has been observed in samples from different manufactured gas plant (MGP) sites where this process option has been implemented. This technology is typically not implemented in saturated conditions because groundwater limits the maximum temperature to the boiling point of water; as water is boiled off, more groundwater flows in. Because the Site has a high water table and the majority of DNAPL is in the saturated zone, extensive dewatering would be required for the duration of treatment to achieve target temperatures.

High-temperature heating is a developing technology for treatment of creosote and coal tar. Full-scale applications of high-temperature heating to creosote or coal tar sites include the Alhambra Site in California, where creosote was treated by heating soil to approximately 650°F using TCH. Treatment reduced the average benzo[a]pyrene concentrations in soil by more than 99 percent. Based on the data collected at the Alhambra Site, the thermal vendor estimated that application of the technology to similarly sized sites (approximately 16,000 cy of treated soil) would cost approximately $380/cy (Baker et al. 2007). High-temperature heating costs at the Quendall Site would likely be higher based on the heterogeneity of subsurface soils (also described under mid-temperature heating) and the significant dewatering that would be required. The cost of implementing this technology, therefore, would be much higher than for *ex situ* treatment options (discussed below) but provide no greater effectiveness. Therefore, this technology has not been carried forward for remedial alternative development.

C2.3.2 In Situ Chemical Treatment

In this technology, chemical oxidants in solution are injected into the subsurface to react with and destroy organic contaminants. Common oxidants include hydrogen peroxide, potassium permanganate, ozone, and sodium persulfate. These chemicals have been shown to destroy a wide range of contaminants, including PAHs, benzene, and other COCs, in soil and groundwater. Full-scale *in situ* chemical oxidation using hydrogen peroxide has been used to treat coal tar DNAPL at MGP sites in New Jersey, New York, Georgia, Michigan, and Wisconsin. Full-scale ozone treatment of coal tar DNAPL has been implemented at two MGP sites in New York. Pilot studies to treat creosote using the oxidant potassium permanganate have reduced mass transfer of creosote constituents to groundwater in the short term, because manganese precipitates form in the vicinity of DNAPL, but they have not shown significant mass reduction or long-term improvements in groundwater quality (Thomson et al. 2008).

This technology's effectiveness is generally limited in heterogeneous and low-permeability soils because of poor distribution of the oxidants, which must contact contaminants directly to be effective. Additionally, high organic matter concentrations in the subsurface consume oxidants and decrease treatment efficiency. The majority of the COC mass at the Site is located within heterogeneous soils containing peat and organic silt layers that would consume oxidants and make the process inefficient. Bench- or field-scale treatability testing would be required prior to design and implementation of this technology.

A review of 13 DNAPL sites where chemical oxidation was applied (primarily chlorinated solvent DNAPL sites, for which chemical oxidation has been applied more frequently than for creosote DNAPL sites) identified an average chemical oxidation treatment cost of $130/cy, which is greater than the cost for thermal treatment. Because it has not been demonstrated to be effective for creosote DNAPL and because potential treatment costs are higher than thermal treatment, this technology was not carried forward for remedial alternative assembly.

C2.3.3 In Situ Stabilization

In this technology, organic and inorganic COCs in soil are physically bound within a stabilized mass (solidification) while chemical reactions between the stabilizing agent and the contaminants reduces contaminant mobility. Potential amendments include bentonite, activated carbon, and cement. Bench testing may be needed to determine an amendment or blend of amendments to achieve performance criteria. Amendments can be mixed with soil *in situ* using large-diameter augers or jet-grouting equipment. Through this process, free-phase DNAPL is reduced to below its residual saturation level by mixing with amendments, which reduce soil permeability and contaminant leachability.

Geotechnical soil properties, such as compressive strength, are often improved by *in situ* stabilization, although solidified soil may complicate installation of future utilities or other subsurface structures. This treatment method does not destroy contaminants and increases the volume of contaminated material. *In situ* stabilization potentially can be applied deeper than excavation at sites with high water tables, such as this Site, and was used at the adjoining JH Baxter site immediately to the north to immobilize similar contaminants. It has frequently been used for source control at Superfund sites, and unit costs typically range from $40 to $60/cy for the depth range of DNAPL at the Site (EPA 2009). Therefore, this technology has been retained for remedial alternative assembly.

C2.4 DNAPL Removal Technologies

DNAPL can either be removed directly as a free-phase product by pumping fluids from wells or trenches or by removing soil or sediment containing DNAPL. Removal and treatment methods for soil and sediment containing DNAPL are discussed in the soil and sediment sections below. This section discusses methods of removing free-phase DNAPL from groundwater. Process options are as follows:

- **Vertical Wells.** Vertical wells can be installed with carefully placed screen sections to maximize DNAPL removal from targeted zones. Wells can include sumps for collecting DNAPL if the underlying confining layer is adequately thick. The main disadvantage of vertical wells is the potential for incomplete fluids capture in heterogeneous soils, as well as limited radius of influence in low-permeability soils. In some cases, this can be overcome by installing vertical wells at multiple levels and spaced closely together. Because vertical wells are a proven technology, they have been retained as a potential DNAPL removal technology.
- **Horizontal or Angled Wells.** Horizontal drilling techniques have been used at some cleanup sites to install non-vertical wells that provide access to areas where the surface is inaccessible to drilling rigs or trench installation. This technology could be applied in the nearshore Quendall Pond area to recover DNAPL; however, angled wells targeted to relatively shallow contamination (as observed in this area) would provide for only minimal additional lateral DNAPL capture compared to vertical wells. Construction of horizontal DNAPL recovery wells is not a proven technology. Therefore, horizontal and angled wells have not been retained.
- **Trenches.** Trenches generally allow more effective capture of groundwater and DNAPL than individual vertical wells by providing an expanded zone of influence (capture). Trenches are typically the preferred method for groundwater collection at sites with heterogeneous subsurface soils and shallow DNAPL occurrences (such as this Site), but constructing DNAPL collection trenches may require significant dewatering, particularly when working adjacent to the lake. Additionally, in areas of stratified DNAPL occurrences (as observed in the Quendall Pond area), trenching could increase DNAPL vertical mobility. Future Site use may limit the use of trenches for DNAPL recovery. Because trenches may be effective and less costly, trenches have been retained as a potential DNAPL removal technology in areas with suitable subsurface stratigraphy.

DNAPL pumped from wells or trenches can either be recovered by itself or with groundwater (total-fluids recovery). Site DNAPL is more viscous than water, flows into wells relatively slowly, and would be most efficiently recovered separately by low-flow or intermittent pumping, likely from a sump constructed in a well or trench, which allows DNAPL in the surrounding soil to drain by gravity and collect in the well. When combined with groundwater pumping, oil-wet soil surrounding the well can become water-wet, limiting DNAPL flow toward the well. A variety of pumping options are available for DNAPL and groundwater under both low-flow and high-flow pumping applications, including above-ground pumps (e.g., peristaltic pumps) and down-well pumps (e.g., electric submersible pumps).

Removal by pumping from wells or trenches can remove DNAPL that is present above residual saturation only. DNAPL present in oil-coated soil would not flow into wells or trenches[3] and would not be treated by this technology. Residual DNAPL can be mobilized for removal and treatment using thermal techniques, as described above.

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[3] A small portion of additional DNAPL could be mobilized under a strong hydraulic gradient induced by total-fluids pumping; however, as described above, passive DNAPL collection is anticipated to be the more efficient method of DNAPL recovery at the Site.

The majority of Site DNAPL is present in thin layers and/or below residual saturation. A DNAPL recovery pilot test was successful at removing approximately 100 gallons of DNAPL from three recovery wells during a 2-year period; however, this is a small fraction (0.2 percent) of the total Site DNAPL mass (estimated to be nearly 500,000 gallons; see Section 4.4.1 of the FS). Therefore, this technology would be potentially effective for supplementing a containment strategy but not for source reduction. Given this application and the heterogeneous Site soils, recovery trenches would be the preferred collection method because they would be less sensitive to heterogeneous soil conditions. Therefore, DNAPL recovery using passive collection and pumping from trenches was retained as a representative process option for this FS.

C2.5 DNAPL *Ex Situ* Treatment Technologies

DNAPL collected from liquid pumping or separated from other waste materials would likely be classified as a hazardous waste based on the high concentrations of PAHs (Washington State persistent dangerous waste WP01) and, in some areas, high concentrations of benzene (Resource Conservation and Recovery Act [RCRA] characteristic hazardous waste D018). The process options for *ex situ* treatment of recovered DNAPL include incineration. If DNAPL is classified as a hazardous waste and recycling/reuse is impractical, it would likely need to be shipped to a hazardous waste treatment facility and incinerated. This is typically a very expensive disposal technology, but the high energy content of DNAPL may reduce the cost. This technology has been carried forward for remedial alternative assembly.

C2.6 DNAPL Disposal Technologies

Recovered DNAPL disposal process options include:

- Recycling of recovered DNAPL; and
- Disposal of recovered DNAPL via off-site incineration (refer also to the previous section).

If available, DNAPL recycling is the preferred and lowest cost method of disposal but may not be practicable because of the potential for hazardous waste classification and the low demand for this product. This technology has been retained for this FS.

In incineration, contaminated material is heated to temperatures above 1,400°F, directly oxidizing and converting volatile organic compounds (VOCs) and semivolatile organic compounds (SVOCs) to carbon dioxide and water. Metals are not treated, though they may be volatilized and the offgas may require treatment. This technology is an EPA presumptive remedy at wood treatment sites and can achieve treatment efficiencies between 90 and 99 percent (EPA 1995). This technology has been retained for this FS.

C3 Soil Technologies and Process Options

C3.1 Soil Institutional Controls

See Section 2.1 (of this appendix) for a description of institutional control technologies and process options effective at preventing exposure to hazardous substances in soil, which are the same as those for DNAPL.

C3.2 Soil *In Situ* Containment

A common method of controlling exposure to contaminated soils is to place an engineered cap over the materials. The long-term cap integrity can be maintained through implementation of appropriate institutional controls. In many cases, the clean cap may be separated from underlying potentially contaminated materials with a marker (e.g., geotextile fabric) indicating the cap boundary.

Process options for soil capping include the following:

- **Permeable Soil Capping.** Placing clean soil on the surface provides a barrier that prevents exposure to underlying soil but allows stormwater to infiltrate. Permeable soil caps implemented without additional measures (e.g., hydraulic controls to limit stormwater infiltration) may not address the soil to groundwater migration pathway in identified source areas. Cap thicknesses of 2 feet are typical in this application, potentially varying based on specific land uses and the presence of existing clean cover materials.
- **Low-Permeability Capping.** A low-permeability cap, constructed of clay or an engineered material, such as asphalt or concrete, would not only prevent exposure to underlying soils, but would also minimize stormwater infiltration through potentially contaminated materials, thereby reducing mobility of contaminants located in the unsaturated soil zone. Engineered materials could also be used in areas requiring a durable surface, such as high-traffic areas.
- **Impervious Capping.** An impervious cap, constructed of clay overlain by a synthetic liner, provides an additional impermeable layer, preventing infiltration to underlying soils and direct exposure, and also controls erosion. A slurry wall may be constructed along the perimeter of the cap to fully contain contaminated material.

Permeable, low-permeability, and impervious caps are proven, effective, and easily implemented, and can be designed to address the Site COCs. Engineered low-permeability and impervious caps are significantly more costly and require more maintenance, but may provide further groundwater mobility controls and may also be compatible with future land uses. Therefore, these three process options have been retained for remedial alternative assembly.

C3.3 Soil *In Situ* Treatment

In situ treatment technologies for soil include the following:

- Interstitial media removal and treatment;
- Thermal treatment;
- Stabilization;

- Chemical treatment; and
- Bioremediation.

These technologies, and available process options, are described below.

C3.3.1 Interstitial Media Removal and Treatment

Interstitial media removal and treatment options include passive soil venting, soil vapor extraction, and soil flushing as discussed below.

Passive Soil Venting. Passive soil venting is a less aggressive version of soil vapor extraction that is usually applied to prevent contaminated soil vapors from migrating into buildings or crawl spaces. In passive venting, soil vapors beneath a building foundation are vented to the atmosphere either through atmospheric pressure changes or by applying a low vacuum with a ventilation fan. Vented vapors can be passed through activated carbon for treatment, if necessary. There are no existing on-Site buildings that are occupied and would require sub-foundation venting, so this has not been retained for the development of remediation alternatives. However, use of this technology may be appropriate under future development scenarios that may include permanent, heated buildings. This potential application may be included as an institutional control for future Site uses.

Soil Vapor Extraction. In soil vapor extraction, a vacuum is applied to subsurface soil to remove soil vapor. Volatile constituents in soil are removed in the vapor stream and are treated above ground. This technology works best on VOCs in homogeneous, permeable soils. It is not effective for SVOCs or metals, and is not applicable to soils below the groundwater table. At the Site, the groundwater table is very shallow, unsaturated soils are highly heterogeneous and often have a low permeability, and much of the contamination identified in the unsaturated zone consists of heavier SVOCs. Therefore, this technology has not been retained for this FS.

Soil Flushing. Soil flushing is an enhancement to groundwater extraction and treatment in which a solution that enhances the solubility of organic constituents is injected into groundwater, passed through contaminated soil to remove contaminants, and then extracted for treatment. Soil flushing is a developing technology that would require bench and field testing prior to design and implementation. It would be potentially applicable to VOCs and SVOCs but not to metals. Surfactants and alcohols are examples of flushing solutions. Field applications of this technology have had mixed results. The effectiveness of soil flushing is limited when applied to heterogeneous soils (such as those at the Site) that cause poor subsurface distribution of the flushing solution and make complete capture of the mobilized contaminants difficult. Incomplete capture of mobilized contaminants at the Site could result in discharge of hazardous substances to Lake Washington. A review of six sites where this technology has been implemented identified an average cost of treatment to be $385/cy, much higher than thermal, chemical, or biological treatment methods (McDade, et. al 2005). Therefore, this technology was not retained for this FS.

C3.3.2 In Situ Thermal Treatment

In situ thermal treatment options include low-, mid-, and high-temperature heating and vitrification as discussed below.

Low-, Mid-, and High-Temperature Heating. *In situ* thermal treatment technologies (as described in Section 2.3.1 of this appendix) are not effective for metals, but potentially are applicable to other COCs in soil as follows:

- Low-temperature heating is low to moderately effective for VOCs and of low effectiveness for SVOCs.
- Mid-temperature heating is highly effective for VOCs and low to moderately effective for SVOCs.
- High-temperature heating is highly effective for both VOCs and SVOCs.

Screening of these technologies for *in situ* DNAPL, including benefits, limitations, and typical costs, is described above and may be applicable for soil. Based on this screening, low-temperature heating is not retained because of its low potential effectiveness, and mid- and high-temperature heating are not retained because of their high cost and difficult implementability compared to other options.

Vitrification. Vitrification involves applying a strong electrical current to the subsurface, heating soil to temperatures above 2,400°F to fuse it into a glassy solid. Organic compounds are destroyed or volatilized by the heating process; volatilized compounds are collected in the offgas and treated. Inorganic compounds are immobilized within the glass. This process would be effective for the Site COCs. Because of the very high energy requirement, particularly in water-saturated soils, this technology is extremely expensive when compared to other soil treatment methods. Although vitrification is equally effective when compared to other high-temperature thermal treatment options (thermal desorption), it is much more expensive than thermal desorption because vitrification operating temperatures are up to three times higher than those required by thermal desorption. This technology was originally designed for handling radioactive waste and has only been implemented at one Superfund site because costs have precluded it as a viable treatment option in other cases (EPA 2009). Therefore, this technology was not retained for remedial alternative assembly.

C3.3.3 In Situ Stabilization

In situ solidification/stabilization for DNAPL is described above and may be applicable for soil. This technology was used at the adjoining JH Baxter Site immediately to the north to immobilize similar contaminants and has frequently been used for source control at Superfund sites. Therefore, this technology has been retained for this FS.

C3.3.4 In Situ Chemical Treatment

In situ chemical treatment technologies include chemical oxidation and electrochemical remediation as discussed below.

Chemical Oxidation. The chemical oxidation process is described in Section 2.3.2 of this appendix and may be applicable for soil. Chemical oxidation is not effective for metals, but potentially is applicable to other COCs in Site soils.

The effectiveness of this technology is generally limited in heterogeneous soils because of poor distribution of the oxidants, which must contact contaminants directly to be effective. Additionally, high organic matter concentrations in the subsurface consume oxidants and decrease treatment efficiency. Because the Shallow Alluvium (which contains the majority of the soil COC mass) contains layers of low-permeability peat and organic silt, which are relatively impermeable and high in organic carbon, applying chemical oxidation to this zone would be costly and inefficient.

A review of 13 sites containing DNAPL at which chemical oxidation was applied identified an average treatment cost of $130/cy, greater than costs for thermal treatment or bioremediation (McDade et al. 2005). Because both bioremediation and thermal treatment potentially are more cost-effective options, chemical oxidation was not retained for this FS.

ElectroChemical Remediation. ElectroChemical Remediation Technology (ECRT) is an innovative technology for destroying organic contaminants *in situ* by applying an alternating current across electrodes placed in the subsurface (EPA 2007). In theory, the applied voltage creates redox reactions that destroy constituents through oxidation-reduction mechanisms. The primary advantage of this technology is that it can treat soil within the unsaturated and saturated zone. The disadvantages are that it has produced mixed results at the field level, and studies indicate that treatment is less effective in soils with high organic carbon content such as those at the Site. Therefore, this technology has not been retained for this FS.

C3.3.5 Bioremediation

Many of the Site COCs, including benzene and naphthalene, can be degraded by native microbial populations. Contaminant biodegradation under natural conditions is one element of natural attenuation. Bioremediation involves adding amendments to the subsurface to enhance *in situ* biological degradation of contaminants. This technology is most effective for VOCs, but is also effective (at a slower rate) for some SVOCs. Bioremediation is least effective for high-molecular weight (5- or 6-ring) PAHs (including benzo[a]pyrene). Bioremediation is not effective for metals; however, changes in groundwater chemistry, such as redox conditions, may cause some metals to form less toxic complexes or become insoluble, precipitating out of solution.

Site VOCs and SVOCs would degrade most efficiently using electron acceptors such as oxygen, nitrate, and sulfate. Oxygen is typically the preferred amendment, but delivery of other electron acceptors is easier under some conditions.

Process options include the following:

- **Amendment Injection.** This process option delivers amendments to the saturated zone. Amendments are typically injected into groundwater and can be used to promote bioremediation of groundwater and saturated-zone soil. Biosparging (adding oxygen to groundwater by injecting air) is typically the most cost-effective bioremediation method for VOC and SVOC contamination.
- **Bioventing.** This process option increases oxygen in the unsaturated zone by extracting soil vapor, similar to soil vapor extraction (SVE). This process draws in atmospheric oxygen, which stimulates microbial growth.

Bioventing is only applicable to the unsaturated zone. Similar to SVE, it would have low effectiveness because of the shallow water table and the fact that most contaminants in unsaturated soils are SVOCs and are not efficiently treated by this process. Therefore, bioventing was not retained.

Similar to other treatment technologies that rely on subsurface distribution of chemicals, bioventing would be inefficient when applied to heterogeneous soils of various permeabilities; however, unlike chemical oxidation, in which oxidants are consumed relatively quickly in the subsurface[4], amendments may diffuse from high-permeability zones into low-permeability zones over time and can stimulate growth beyond the injection zones.

A review of 11 sites containing DNAPL where bioremediation was applied determined that costs for bioremediation range widely, from $2 to $225/cy, but that the average treatment cost was $29/cy, cheaper than chemical oxidation, surfactant flushing, or thermal treatment (McDade et al. 2005)[5]. *In situ* bioremediation is an EPA presumptive remedy for wood treatment sites (EPA 1995). Biodegradation is ongoing at the Site, has been widely demonstrated, and could be implemented as a polishing technology for other more effective technologies. Therefore, bioremediation via amendment injection was retained for remedial alternative assembly.

C3.4 Soil Removal Technologies

Contaminated soils can be effectively removed by excavation. Excavators, backhoes, and other conventional earth moving equipment are the most common equipment used to remove contaminated soil from upland areas. Below the water table, dewatering may be required to use soil excavation equipment. Alternatively, dredging equipment (see Section 5, Sediment Technologies and Process Options, of this appendix) could be used to remove soil 'in the wet.' Contaminated soil excavation is a commonly implemented technology and has been retained for remedial alternative assembly.

C3.5 *Ex Situ* Soil Treatment Technologies

Soil may be treated using physical, thermal, or biological technologies. These technologies and process options are described below.

C3.5.1 *Ex Situ Physical Treatment*

Ex Situ physical treatment options include physical separation and solidification/ stabilization as discussed below.

Physical Separation. The volume of excavated contaminated materials can be reduced by physically separating the materials into two or more fractions that can be handled separately. For example, cobbles can be screened from contaminated soil and beneficially used. However, large gravels or cobbles are not prevalent in the upland area of the Site. Therefore, there is little or no benefit from applying physical separation; therefore, this technology has not been retained for this FS.

Solidification/Stabilization. Similar to *in situ* solidification/stabilization for DNAPL described above, *ex situ* solidification/ stabilization is performed *ex situ* to excavated soils using a pug mill or similar equipment to blend soil with amendments. Depending on the amount of amending agent used and/or the type of amending agents, the end product

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[4] Some oxidants, such as permanganate, can persist at some sites for a long period of time. However, natural oxidant demand at the Quendall Site is expected to rapidly consume injected oxidants.

[5] This study reviewed primarily sites with chlorinated solvent DNAPL.

may take on the form of a quasi-soil/concrete material that could later be used as bulk fill or a solid mass that could be used as building blocks or tiles (FRTR Website 2012).

Solidification/stabilization is a presumptive remedy at wood treatment sites, but only for inorganic constituents (EPA 1995). This technology would have similar effectiveness to *in situ* stabilization (which is retained) but is more expensive, with costs typically ranging from $70 to $145/cy (EPA 2009). Therefore, this technology was not retained for this FS.

C3.5.2 Ex Situ Thermal Treatment

Ex situ thermal treatment options include thermal desorption, vitrification, and incineration as discussed below.

Thermal Desorption. Low-temperature thermal desorption involves heating soils to temperatures between 200°F and 600°F until VOCs and SVOCs, such as benzene and naphthalene, evaporate. Exhaust gases produced by the process are typically combusted. This technology is effective for VOCs and SVOCs, achieving 90 to 99.7 percent destruction efficiencies for PAHs (EPA 1999), but is not effective for metals. It is a presumptive remedy for wood treatment sites (EPA 1995).

Thermal desorption systems can be designed to operate without producing liquid or solid secondary wastes, to meet clean air standards, and to achieve very low concentrations of residual constituents in soil. Limitations include high energy requirements for treating wet soils, difficulty in completely treating soils containing high levels of organics (such as the peaty Site soils), and the need to obtain permits for treatment of offgas (typically via incineration) generated from the on-site thermal desorption system. Thermal desorption may be accomplished on site with a mobile treatment unit or off site at a permanent treatment facility. Treatment costs (including excavation, backfilling, and sampling) typically range between $78 and $110/ton for on-site treatment and approximately $100 to $200/ton for off-site treatment (EPA 1999).

Compared to off-site landfill disposal, thermal desorption is typically more expensive than disposal at a Subtitle D (non-hazardous waste) landfill, but has the advantage of providing contaminant treatment and destruction rather than containment. This technology is typically less expensive than disposal at a hazardous waste landfill (for medium to large quantities of soil) and less expensive and/or more effective than other *ex situ* treatment options. Therefore, this technology has been retained in the FS as a representative process option for *ex situ* treatment of contaminated soils.

Vitrification. Vitrification involves the application of a strong electrical current to heat sediment to temperatures above 2,400°F, fusing it into a glassy solid. Organic compounds are destroyed or volatilized by the heating process; volatilized compounds are collected in the offgas and treated. Inorganic compounds are immobilized within the glass. Because of the very high energy requirement, particularly in water-saturated sediments, this technology is extremely expensive when compared to other treatment methods. Although vitrification is equally effective in remediating organic compounds as other high-temperature thermal treatment options (thermal desorption), it is much more expensive than thermal desorption because vitrification operating temperatures are up to three times higher than those required by thermal desorption. Therefore, vitrification has not been retained for soil in this FS.

Incineration. In incineration, contaminated soil is heated to temperatures above 1,400°F, directly oxidizing and converting VOCs and SVOCs to carbon dioxide and water. Metals are not treated, though they may be volatilized and the offgas may require treatment. This technology is an EPA presumptive remedy at wood treatment sites and can achieve treatment efficiencies between 90 and 99 percent (EPA 1995). However, this technology is relatively expensive, with typical costs up to $400/ton (EPA 1999) for on-site treatment and up to $900/ton for transport[6] and off-site treatment. Permitting on-site units can be costly and implementation can be difficult because of public opposition. This technology was not retained based on its high cost and the availability of other effective and cheaper treatment options such as thermal desorption.

C3.5.3 Ex Situ Chemical/Physical Treatment

Ex Situ chemical/physical treatment options include soil washing and solvent extraction as discussed below.

Soil Washing. In soil washing, soil is put in contact with an aqueous solution to remove contaminants from the soil particles. The suspension is often also used to separate fine particles from coarser particles, allowing beneficial use of the coarser fraction (if sufficiently clean). The aqueous solution can contain surfactants or other additives to promote contaminant dissolution. Soil washing has rarely been implemented in the United States and is typically more expensive than thermal desorption, with an average cost of approximately $170/ton (EPA 1999). It has limited effectiveness for removing strongly hydrophobic chemicals such as PAHs, particularly from soils with a high organic content, and is not typically effective when soil is composed of large percentages of silt or clay (EPA 1999), as are Site soils. Therefore, this technology was not retained for this FS.

Solvent Extraction. Solvent extraction is a variant of soil washing in which an organic solvent (rather than an aqueous solution) is put in contact with the soil to remove contaminants. This technology is more effective than soil washing at removing hydrophobic organic compounds such as PAHs, but is more expensive to implement because the solvent must be carefully controlled, collected, treated, and recycled. This technology has many of the same limitations as soil washing and would not be cost competitive or offer better treatment than thermal desorption. Therefore, this technology was not retained for this FS.

C3.5.4 Ex Situ Biological Treatment

Contaminant biodegradation by indigenous soil microbes can be enhanced by amending excavated soil with nutrients, moisture, and oxygen (typically provided by mixing). Process options for biological treatment include the following:

- **Landfarming/Composting.** Contaminated soil is spread out in a lined area and regularly tilled and amended with moisture and nutrients. Unit costs for treatment by this method are approximately $75/cy (EPA 1999).
- **Biopiles.** Contaminated soil is amended with nutrients and stockpiled. Unit costs for treatment by this method are approximately $100 and $200/cy (EPA 1999).

[6] Limited off-site incineration options exist, with no off-site incineration facilities in the Pacific Northwest.

- **Bioreactor.** Contaminated soil is mixed in a vessel with nutrients and water to make a slurry. Unit costs for treatment by this method are approximately $216/cy (EPA 1999).

Ex situ biological treatment methods have limited effectiveness for high molecular weight PAHs, are slower than other treatment technologies, and require significant space to implement (EPA 1999). These technologies have lower effectiveness with similar or higher costs than other treatment options. Therefore, *ex situ* bioremediation of Site soils was not retained for this FS.

C3.6 Soil Disposal Technologies

Excavated soils may be either disposed of directly or treated, using one or more of the technologies retained in the analysis above, and then disposed of. At a minimum, saturated soils would likely require dewatering before disposal. Soil disposal options are described below.

C3.6.1 On-Site Beneficial Use

Excavated soils exceeding applicable cleanup standards may potentially be used on Site if they meet or can be treated to meet applicable cleanup standards. Process options for on-Site beneficial use consist of:

- **Sand/Aggregate Reclamation.** Particle separation of excavated material with high sand content for use as concrete aggregate or general upland fill.
- **Topsoil Feedstock.** Blending of excavated material with organics for use as non-organic topsoil feedstock.

On-Site reuse may be appropriate for excavated soils, depending on COC concentrations and future Site use, and is of moderate relative cost. Both sand/aggregate reclamation and topsoil feedstock process options have been retained as representative on-Site beneficial use process options.

C3.6.2 On-Site Confined Disposal

Excavated soils exceeding applicable cleanup standards can be disposed of on Site within a specially designed upland confined disposal facility (CDF). On-Site confined disposal can be less costly than off-Site confined disposal but requires long-term on-Site management of contaminated materials.

An upland on-Site CDF may be appropriate for disposal of excavated soils, depending on COC concentrations and future Site use, and is of moderate relative cost. The on-Site upland confined disposal process option has been retained as the representative on-Site confined disposal process option.

C3.6.3 Off-Site Landfill Disposal

Contaminated soils may be transported to an off-Site, permitted disposal facility. The proper disposal facility will depend on whether the soil is classified as a non-hazardous or hazardous waste. No listed RCRA wastes have been identified on the Site (Ecology 2002). Potentially hazardous waste classifications based on soil characteristics include the following:

- **Washington State Persistent Dangerous Waste (WP01).** Soil is classified as WP01 if total PAH concentrations exceed 1 percent. Based on analytical data collected during the RI, most DNAPL-containing soil contains less than 1 percent total PAHs (see Table 4.2-1 of the RI Report). Furthermore, soil or sediment containing DNAPL that is removed is likely to be blended with cleaner soils during removal and processing, further lowering the total PAH concentration. Therefore, most soil generated during a remedial action at the Site is not expected to be classified as WP01.
- **RCRA Hazardous Waste (D018).** Soil is classified as D018 if benzene toxicity characteristic leaching procedure (TCLP) concentrations exceed 0.5 milligrams per liter (mg/L; which is approximate, if benzene is leached during the TCLP test, to a soil concentration of 10 milligrams per kilogram [mg/kg]). Based on analytical data collected during the RI, soil containing DNAPL in the Quendall Pond area potentially exceeds this value, and could exceed even if blended with a reasonable volume of clean soil during excavation.

Other potentially hazardous constituents detected at the Site (including cresol, arsenic, and lead) have not been detected at concentrations potentially exhibiting a hazardous waste characteristic.

Most contaminated Site soils will likely be characterized as non-hazardous solid wastes. However, some wastes (including highly concentrated DNAPL-containing soil, or DNAPL-containing soils in the vicinity of Quendall Pond) could be classified as hazardous wastes.

Non-hazardous solid wastes would be shipped via truck and/or railcar to a Subtitle D facility, such as the Klickitat County Landfill in Roosevelt, Washington. This disposal method provides for secure, long-term containment of non-hazardous solid wastes. Disposal costs at this facility can vary with quantity and season but currently average approximately $45/ton.

Some Site soils could be characterized as an RCRA hazardous waste or state-only dangerous waste based on either the presence of benzene (in coal tar-contaminated soil) or high PAH concentrations. Soils characterized as hazardous waste, but at concentrations less than ten times the Universal Treatment Standards, could be shipped via truck and railcar to a Subtitle C facility, such as the Waste Management Landfill in Arlington, Oregon. Disposal costs at this facility typically range from approximately $100 to $190/ton. Because off-Site disposal effectively removes contaminants from the property and places them in a secure containment facility, and because it is cost competitive when compared to on-Site treatment technologies (particularly for relatively small quantities of materials), these disposal options have been retained for remedial alternative assembly.

C4 Groundwater Technologies and Process Options

C4.1 Groundwater Institutional Controls

Institutional controls limit access to contaminated groundwater and may consist of legal restrictions such as use limitations recorded on the property deed. Process options for institutional controls include:

- Deed restrictions restricting use of groundwater for drinking; and
- Deed restrictions restricting use of groundwater wells.

These institutional controls can be effective and implementable under a wide range of conditions and generally apply to the entire Site. Consequently, these institutional control process options were retained as a representative institutional control process options.

C4.2 Groundwater Monitored Natural Attenuation

Natural attenuation is the reduction of COC groundwater concentrations through a combination of naturally occurring physical, chemical, and/or biological processes. Some natural processes (e.g., sorption of hydrophobic organic contaminants to organic carbon in soil) act as containment mechanisms and others (e.g., biodegradation of contaminants by native bacteria) act as *in situ* treatment mechanisms.

Natural attenuation of Site COCs (primarily coal tar/creosote constituents) has been widely documented at similar sites, and biodegradation of key COCs such as benzene and naphthalene has been documented at the Site (see Section 6 of the RI Report). The consistency of Site-specific biodegradation rates across the Site, as well as their similarity to literature information, provides support that natural attenuation of dissolved-phase groundwater contaminants is an important process to consider during development of remedial alternatives.

As a general response action, monitored natural attenuation would include monitoring to document the presence and effectiveness of natural processes in removing or containing Site COCs. Measures to enhance natural processes are considered under the *in situ* treatment options. Potential technologies applied under monitored natural attenuation include further characterization and predictive modeling of natural attenuation processes, and performance monitoring to verify model predictions.

Natural attenuation will likely be an important mechanism affecting contaminant fate and transport under any general response action. While monitored natural attenuation may not be effective at achieving the RAOs as a stand-alone technology, this technology is highly implementable at the Site. Therefore, this technology was retained as a possible supplemental polishing technology to be combined with other groundwater remediation technologies.

C4.3 Groundwater *In Situ* Containment

Methods of groundwater containment include impermeable vertical barriers, groundwater pumping, and stormwater controls. These technologies and process options are described above for DNAPL and their specific applications to groundwater are further discussed below.

C4.3.1 Impermeable Vertical Barriers

Vertical barrier technologies and process options, described in Section 2.2 of this appendix, may be applicable for controlling the material movement on contaminated groundwater. To prevent groundwater mounding behind the barrier, which would result in flow of contaminated groundwater beneath or around the barrier, a groundwater pumping system would likely need to be implemented. To reduce the amount of pumping required, the vertical barrier could be installed to completely encircle the area being treated. Process options include sheet pile walls, slurry walls, and grout curtains, which are described above for *in situ* treatment of DNAPL.

Vertical barriers are commonly implemented as part of containment remedies at Superfund sites. They can also be used to facilitate construction of treatment remedies, such as excavation, that require dewatering. As described for DNAPL above, slurry walls constructed by excavating trenches are likely the most reliable and cost-effective process option and have been retained as the representative process option for impermeable vertical barriers.

C4.3.2 Groundwater Pumping

Migration of dissolved groundwater contaminants can be controlled by pumping groundwater from vertical wells or trenches, creating a capture zone within which groundwater flows toward the capture point. This technology can be applied for the groundwater COCs. The effectiveness of this technology to completely capture contaminated groundwater is often limited at sites with heterogeneous soils (such as the Site). It would not be effective at capturing groundwater beneath the lake. Because of subsurface heterogeneities and the close proximity of Lake Washington, groundwater pumping would likely need to be implemented with vertical barriers to contain the contamination plume. Short-term groundwater pumping may be a component of another technology, such as dewatering to support soil excavation. Because of its common application to other sites and its potential short-term applications, groundwater pumping was retained for remedial alternative assembly.

C4.3.3 Stormwater Controls

Migration of groundwater contaminants can be controlled by modifying hydraulic gradients influenced by stormwater infiltration. Process options for stormwater controls include:

- **Targeted Infiltration.** Creation of a hydraulic barrier by collecting and infiltrating stormwater and forming a local groundwater ‘mound.’
- **Reduced Infiltration.** Reduce localized infiltration and seepage of stormwater in impacted areas along the shoreline by implementing hydraulic controls, such as an impermeable shoreline cap.

Implementation of targeted infiltration may be limited because seasonal variability of Site groundwater elevations. Reduced infiltration through impermeable capping is moderately effective and implementable under a variety of future Site uses; therefore, reduced infiltration has been retained as the representative stormwater control process option.

C4.4 Groundwater *In Situ* Treatment

In situ groundwater treatment technologies include permeable reactive barriers, chemical treatment, and bioremediation, which are described below.

C4.4.1 Permeable Reactive Barrier

A permeable reactive barrier can be used to limit the migration of dissolved groundwater contaminants by passively treating groundwater as it flows through the barrier. The process option for permeable reactive barriers consists of a sorptive/reactive wall. A sorptive/reactive wall consists of a trench excavated in the upland and backfilled with permeable reactive materials. As groundwater flows through the barrier, permeable materials within the barrier sorb dissolved-phase constituents and can promote biodegradation. Sorptive/reactive walls materials applicable to coal tar/creosote Site COCs include activated carbon, organoclay, and materials with a high organic content, such as wood debris. Amendments to increase biodegradation may include calcium nitrate or other electron acceptors.

A permeable treatment wall to treat arsenic in groundwater using granular iron was installed, using excavation and bioslurry displacement, to a depth of 22 feet along the shoreline on the adjacent Conner Homes property. Installation of deeper treatment walls is possible but would likely require different techniques depending on the amendment. Permeable treatment walls potentially are effective at preventing upland groundwater contamination from discharging to Lake Washington; however, this technology would not address contaminants that have already migrated beneath the lake. Because of its potential effectiveness to treat upland groundwater and its proven implementability, this technology has been retained for remedial alternative assembly.

C4.4.2 In Situ Chemical Treatment

In situ chemical treatment technologies and process options are described in FS Section 5.4.3.4 and in Section 2.3.2 of this appendix.

C4.4.3 Bioremediation

The two process options for bioremediation of groundwater include the following:

- **Amendment Injection.** Described in Section 3.3.5 above.
- **Biosparging.** During biosparging air is bubbled into groundwater. This technology is generally the most cost-effective method of delivering oxygen to the subsurface, but its effectiveness can be limited in heterogeneous soils that are not conducive to air distribution.

Bioremediation is generally not effective for metals, but is potentially applicable to other Site groundwater COCs. Biodegradation is most effective for VOCs and least effective for high-molecular weight (5- or 6-ring) PAHs. Changes in groundwater chemistry associated with bioremediation may cause metals to form less toxic metal complexes or become insoluble by precipitating out of solution. Bioremediation is less costly than other *in situ* technologies, such as chemical oxidation.

Biodegradation of Site COCs, which has been demonstrated at other similar sites, could be implemented as a polishing technology when combined with other technologies. Either of these process options may be appropriate depending on where the technology is applied. For example, biosparging is best suited to applications in the Deeper Alluvium. Therefore, both process options for bioremediation were retained for remedial alternative development.

C4.5 Groundwater Removal Technologies

Groundwater can be removed from the subsurface by pumping fluids from wells or trenches. A variety of pumping options are available for groundwater but down-well pumps (e.g., electric submersible pumps) are most commonly used. Groundwater may be pumped from vertical wells, horizontal or angled wells, or trenches.

Groundwater removal for treatment has been implemented and is ongoing at many Superfund sites. While it would not be expected to adequately reduce source area concentrations for many Site COCs that have low solubility (particularly cPAHs), it could be used as a polishing technology when combined with other technologies. Because of their common use and potential application to the Site, groundwater pumping vertical wells and trenches are retained for remedial alternative development.

C4.6 Ex Situ Groundwater Treatment Technologies

Potentially applicable treatment technologies for extracted groundwater are described and evaluated below. Groundwater would not need treatment if it meets discharge requirements (e.g., if minimally impacted groundwater is extracted as a containment measure).

C4.6.1 Physical/Chemical Treatment

Physical/chemical treatments include adsorption, air stripping, and advanced oxidation processes, which are described below:

- **Adsorption.** Adsorption of dissolved organic contaminants is one of the most widely used water treatment technologies. In this technology, contaminated groundwater is passed through a bed of granulated media where contaminants sorb to the surface of the sorbent, reducing the concentration of COCs in the bulk liquid phase. Activated carbon adsorption is effective and widely used for VOCs and SVOCs. Arsenic is often treated using activated alumina, iron oxides, or greensand. Arsenic treatment using sorption is typically less expensive than other methods if the volume to be treated is less than roughly 1 million gallons per day (EPA 2002). Disadvantages of adsorption include the need to periodically replace and regenerate or dispose of the used media. Adsorption is typically the most cost-effective means of treatment for VOCs, SVOCs, and metals. Because of its proven effectiveness, this treatment technology has been retained as a representative process option in combination with groundwater removal technologies.
- **Air Stripping.** In air stripping, contaminated groundwater and air typically are passed counter-currently through a tower, and volatile contaminants (such as benzene and, to a lesser extent, naphthalene) are transferred from the water to the air. The contaminant-laden air is usually treated by activated carbon and then discharged to the atmosphere. Air stripping can be cost-effective for volatile compounds such as benzene, but it is typically not effective for less volatile compounds such as PAHs. Air stripping is not effective for arsenic. Treatment efficiencies for air stripping are generally less than those for activated carbon, and

air stripping may require water polishing by activated carbon for some discharge options. For treatment of water with high VOC concentrations, this technology may be a cost-effective step in a treatment train. Therefore, this technology has been retained as a representative process option in combination with groundwater removal technologies.

- **Advanced Oxidation Processes.** A number of technologies exist that involve adding chemicals that directly oxidize organic groundwater contaminants. Process options include ozonation, hydrogen peroxide (with or without catalysts such as Fenton's Reagent or ultraviolet light), and permanganate. These technologies can effectively destroy organic chemicals, but capital and operation and maintenance costs are significantly higher than treatment by activated carbon or air stripping. They are not effective to treat arsenic. Therefore, this technology has not been retained for this FS.

C4.6.2 Biological Treatment

Biological treatment consists of contaminant destruction by passing contaminated groundwater through a biological reactor in which a contaminant-degrading microbial culture is maintained, generally by adding nutrients and oxygen, and controlling temperature, pH, and other parameters. Types of biological reactors include bioslurry reactors, fixed-film bioreactors, and constructed wetlands.

Biological treatment is potentially highly effective for treatment of Site groundwater containing VOCs; however, the treatability of recalcitrant COCs (particularly cPAHs) would have to be demonstrated in bench-scale and/or pilot tests. Because biological treatment is likely to be effective for treating Site groundwater and is technically implementable, it has been retained as an *ex situ* representative process option.

C4.7 Groundwater Disposal Technologies

Potential groundwater disposal methods are described and evaluated below. Some disposal methods may require pre-treatment depending on the quality of the extracted groundwater. Inclusion of these technologies in remedial alternatives could occur if short-term dewatering is required as part of construction.

C4.7.1 Off-Site Management

Off-site groundwater disposal process options include discharge to sanitary sewer and discharge to surface water as discussed below.

Discharge to Sanitary Sewer. In this disposal option, recovered groundwater would be discharged to the local sanitary sewer system. Groundwater pre-treatment may not be required if COC concentrations meet discharge criteria. Water containing high solids concentrations (e.g., from construction dewatering) would likely need to be passed through a settling tank or filter to meet discharge requirements. Fees for groundwater disposal to the sanitary sewer are based on the volume discharged, and periodic chemical and physical discharge monitoring is typically required. Allowable discharge volumes may be limited, particularly in the wet season, by the sewer system's capacity. Because this option may allow groundwater discharge without substantial on-Site treatment, it has been retained.

Discharge to Surface Water. In this disposal option, recovered groundwater would be discharged to Lake Washington surface waters. A National Pollutant Discharge Elimination System (NPDES) permit would likely be required for discharges. Water discharged to surface water would have to meet strict water quality requirements and would likely require treatment before discharge; however, no discharge fee (besides permitting fees) would be incurred. This technology has been retained.

C4.7.2 On-Site Management

Extracted groundwater may be discharged on Site via reintroduction to groundwater. Process options for reintroduction to groundwater include infiltration galleries or injection wells. On-Site reintroduction to groundwater is often the preferred disposal method for water generated during construction at large sites, such as the Quendall Site, when practicable. Reintroduction to groundwater as a disposal method is potentially effective, implementable, and cost-effective; therefore, it has been retained as the representative on-Site management process option.

C5 Sediment Technologies and Process Options

C5.1 Sediment Institutional Controls

Institutional controls limit access to contaminated material and may consist of physical restrictions, such as public advisories on fish consumption, or legal restrictions, such as use limitations recorded on the property deed. Process options for institutional controls include:

- Advisories on harvesting fish or shellfish typically implemented and enforced by the local health department.
- Monitoring and notification of waterway users to restrict specific activities to protect the remedy (e.g., restrictions on anchorage within the areas that are capped; restrictions on grounding of small vessels on the shoreline and on vessel draft, horsepower, speed, and time in area; and restrictions on piling placement or removal through cap, or other potential in-water construction/structures). Easements or restrictive covenants to limit activities that may damage the remedy or increase the potential for exposure. These easements or restrictive covenants can be placed on privately-owned aquatic lands or on state-owned aquatic lands through a long-term agreement with the Washington Department of Natural Resources (DNR).

These institutional controls are potentially effective at preventing exposure to hazardous substances and could be implemented under a wide range of conditions. However, institutional controls would not meet RAOs alone. Consequently, these institutional control process options were retained as a representative institutional control process options for combination with active remedial technologies and to protect the selected remedy. These institutional controls are considered applicable to the alternatives with a cap remedy. In addition, for alternatives with a dredging component, short-term fish consumption advisories may be required due to the potential for short-term water quality and fish-tissue impacts during dredging. A remedy including sediment institutional controls will need to be designed to reduce conflicts or restrictions on Tribal treaty fishing rights or other treaty protected rights such as anchorage of Tribal fishing vessels or access to aquatic resources. The combination of monitoring, maintenance, and institutional controls; formal 5-year reviews; and contingency actions (if required) are considered adequate for ensuring remedy integrity.

C5.2 Sediment Monitored Natural Recovery

Natural recovery is the reduction in sediment COC concentrations through a combination of naturally occurring physical, chemical, and/or biological processes. Some natural processes (e.g., sedimentation or sorption of hydrophobic organic contaminants to organic carbon in soil) act as containment mechanisms, while others (e.g., biodegradation of contaminants by native bacteria) act as *in situ* treatment mechanisms.

C5.2.1 Monitored Natural Recovery

As a general response action, monitored natural recovery (MNR) provides monitoring to document the presence and effectiveness of natural processes in removing, reducing the risk, or containing Site COCs. The key difference between monitored natural attenuation (MNA) for groundwater and MNR for sediment is in the type of processes being relied upon to reduce risk. Transformation of contaminants, including biodegradation, is usually the major attenuating process for contaminated groundwater. However, often these processes are too slow for the persistent contaminants in sediment for remediation in a reasonable timeframe. Natural sedimentation is the process most frequently relied upon for MNR (EPA, 2005).

Potential activities completed under MNR include the following:

- Further characterization and predictive modeling of natural recovery processes, including isolation and mixing through natural sedimentation.
- Ongoing monitoring of sediment concentrations and toxicity of surface sediments.

MNR may not be effective at achieving the RAOs as a stand-alone technology, but this technology is highly implementable at the Site. Therefore, this technology was retained as a possible supplemental polishing technology to be combined with other sediment remediation technologies.

C5.2.2 Enhanced Natural Recovery

Deposition of clean sediment plays a role in the natural recovery of contaminated sediments. Enhanced Natural Recovery (ENR) is a remedial approach that enhances MNR by adding a thin layer of clean sediment layer over impacted sediment (i.e., thin-layer placement). The acceleration can occur through several processes, including increased dilution through bioturbation of clean sediment mixed with underlying contaminants. Thin-layer placement is typically different than *in situ* isolation caps because it is not designed to provide long-term isolation of contaminants from benthic organisms. ENR has been implemented as part of a remedy at similar sites. For instance, ENR has been implemented successfully as a component of the larger remedial effort at the creosote contaminated Wyckoff/Eagle Harbor Site on Bainbridge Island (ENVIRON and SPAWAR, 2009). Specifically, the thin layer cap has remained stable during 10 years of monitoring. Therefore, ENR has been retained for remediation of contaminated sediment.

C5.3 Sediment *In Situ* Containment

Engineered caps as an *in situ* containment technology, described for soil above, may be effective for isolating COCs in sediment. Cap monitoring results at other Puget Sound region sites have shown that capping can provide an opportunity for effective and economical sediment remediation without the risks involved in removing contaminants by dredging (Sumeri 1996). Sediment capping has been applied as a component of site remediation at a significant number of contaminated sediment sites (USEPA 2005). Recent demonstrations of reactive capping techniques have also been effective in providing additional protection through enhanced adsorption of contaminants. Capping process options are described below.

C5.3.1 Engineered Sand Cap

An engineered sand cap (typically up to 3 feet thick) can be designed to effectively contain and isolate contaminated sediments from the biologically active surface zone. The cap can be designed to be thick enough and of sufficient grain size to maintain its integrity under reasonable worst-case environmental and land use conditions. A sediment cap system's surface layers would likely be constructed of clean sand and could be placed by a number of mechanical and hydraulic methods. Engineered caps may also include erosion protection or stability layers such as geosynthetics or armoring materials. Armored caps (e.g., with a gravel surface) may be potentially appropriate for consideration in sediment areas with high potential for disturbance (e.g., areas likely to experience propeller wash).

Sediment capping is a proven technology to prevent exposure to contaminated sediments and could be implemented at the Site. Engineered sand caps are relatively cost-effective remediation technologies. Therefore, this process option has been retained for containment of contaminated sediment.

C5.3.2 Post-Dredge Residuals Cap

Recent research focused on evaluating contaminant concentrations of the post-dredge sediment surface indicates that approximately 2 to 11 percent of the mass of solids dredged during the last dredge production cut accumulates as a post-dredge residual layer (Bridges et al. 2010). The research further indicates that additional "cleanup" passes are inefficient in dealing with the generated residuals layer and other management approaches are required. One increasingly common and successful approach is the placement of a post-dredge residuals cap. The purpose of the cap is to provide a reduction in exposure to the residual contamination layer. Because post-dredge residuals caps are effective management solutions, this process option has been retained for containment of contaminated sediment.

C5.4 Sediment *In Situ* Treatment

In situ treatment methods applicable to sediment remediation generally rely on physical, chemical, or biological processes to destroy or immobilize contaminants or reduce toxicity.

C5.4.1 Physical/Chemical Treatment

Physical/chemical treatment options include permeable reactive capping, electrochemical remediation technology (ECRT), and stabilization as discussed below.

Permeable Reactive Capping. This technology could be used in targeted areas where DNAPL or sheens are an issue. In permeable reactive capping, a permeable cap is placed above contaminated sediments, and a material (organoclay or activated carbon) is placed within the sediment cap to sorb NAPL and/or dissolved-phase constituents, limiting migration into overlying sediment porewater and surface water. In certain applications, reactive caps may lose their effectiveness when the reactive material becomes saturated. Therefore, for continued effectiveness, a reactive cap should be designed such that one or more of the following design goals are achieved:

- A sufficient volume of reactive material is added such that its operating lifetime is longer than the projected restoration timeframe; or
- A mechanism to allow for reactive layer replacement is incorporated into the design.

Typical reactive capping media include granular activated carbon (GAC), organoclay, or apatite. The type of reactive media depends on the site COCs. GAC or lower cost coal or coke products are typically used to control dissolved-phase organic compounds. Apatite is used for metals. organoclay is manufactured by replacing cations in layered clays, such as bentonite, with cationic organic compounds, such as quaternary ammonium compounds (QACs), to create an organic phase along the surface of each layer in the molecular lattice. Organoclay effectively controls NAPL and has been installed to control NAPL at several sediment sites.

The Reactive Core Mat® (RCM) developed by CETCO™ uses a reactive material (e.g., organoclay, GAC, or apatite) within a geotextile envelope to provide capacity for contaminant sequestration (e.g., NAPL, organics, or metals) in a thin, rolled product that is readily transported and deployable. RCMs are appropriate for a cap of less thickness than a traditional bulk cap and have a significantly lower weight than bulk caps. Additional benefits of RCMs are their ease of installation, stability, and physical isolation.

Over the last ten years, reactive caps have been installed as full-scale remedies at numerous contaminated sediment sites in the United States, including:

- McCormick and Baxter Creosoting Co. Superfund Site, Portland, Oregon: Bulk organoclay Cap and organoclay RCM;
- Zidell Marine Corporation Sediment Cap, Portland, Oregon: RCM with GAC and apatite;
- Port of Portland Nearshore Cap, Portland, Oregon: Bulk Organoclay;
- Pine Street Canal Superfund Site, Burlington, Vermont: Organoclay RCM;
- Harbor Point Former MGP, Utica, New York: Organoclay RCM;
- Former Salem Massachusetts Manufactured Gas Plant (MGP), Salem, Massachusetts: Organoclay RCM;
- Former MGP, Everett, Massachusetts: Organoclay RCM;
- Bridgeport Rental and Oil Services (BROS) Superfund site, Logan Township, New Jersey: Organoclay RCM;
- Former Gautier Oil Company (CSX) Site, Gautier, Mississippi: Organoclay RCM;
- Stryker Bay St. Louis River/Interlake/Duluth Tar Superfund Site, Duluth, Minnesota: GAC RCM;

- Former Cresote Wood Treating Site, Escanaba, Michigan: Organoclay RCM and Bulk organoclay in a permeable reactive barrier; and
- Grand Calumet River - West Branch, Reach #3, Hammond, IN: GAC RCM.

Reactive caps or RCMs are designed to allow flow of groundwater or porewater through the cap. In addition, organoclay RCMs have been shown to be effective for control of NAPL or sheen at sites with ebullition, including the Pine Street Canal Superfund Site and the McCormick and Baxter Creosoting Co. Superfund Site. The organoclay sorbs/strips NAPL from NAPL-coated gas bubbles so that the bubbles do not transport NAPL beyond the reactive cap layer. For instance, at the McCormick and Baxter Superfund Site in Portland, Oregon, gas bubbles were associated with sheen prior to capping. After installation of the RCM, gas bubbles were still observed; however, there was no longer a sheen associated with the bubbles (Bullock 2007).

Although not applicable for this Site, an innovative application of reactive materials is to physically mix the reactive material with sediments to allow treatment of a thickness of sediment (EPA, 2013). Reactive materials have also been applied for upland sites or on shorelines in both bulk and as RCMs to line DNAPL collection trenches or in permeable reactive barriers. Reactive cap technology has been retained as a process option for *in situ* sediment treatment.

ElectroChemical Remediation Technology (ECRT). The ECRT process option is described in Section 5.3.2.3 of the FS and Section 3.3.4 of this appendix. This technology has been field-scale demonstrated by Weiss Associates Electrochemical Remediation Technologies and Lynntech, Inc., at three sites in the United States: the Duluth/Superior Harbor Superfund Site in Minnesota; the Georgia Pacific Remediation Site in Bellingham, Washington; and the Naval Air Weapons Station in Point Magu, California. In spite of several successful demonstrations in Europe, the projects in the United States were unable to yield favorable results. ECRT was not retained as a process option for *in situ* sediment treatment.

Stabilization. This technology is generally described in Section 2.3.3 above. In the aquatic environment, this process option is applicable to relatively coarse-grained, homogeneous sediment with lower concentrations of contamination and minimal free product present. The Site sediments are typically fine and in heterogeneous deposits. In addition, stabilization of aquatic sediments *in situ* has not been demonstrated to be effective in the long term. Therefore, this process option has not been retained for *in situ* sediment treatment.

C5.4.2 Bioremediation

Described in Section 3.3.5 of this appendix, bioremediation may be effective for reducing COC concentrations in sediment. The bioremediation process option for sediment is amendment injection.

Bioremediation of sediments *in situ* (e.g., via amendment injection) is an innovative technology and may not meet RAOs when implemented alone, but may be effective when combined with other technologies and can potentially be implemented under a variety of Site conditions. Therefore, amendment injection was retained for sediment for future consideration as a potential polishing technology, but not as a stand-alone application.

C5.5 Sediment Removal Technologies

C5.5.1 Excavation

Long-reaching excavators positioned from upland staging areas could be used to remove contaminated sediment. Dry excavation of nearshore sediments may also be facilitated through the installation of temporary cofferdams and the subsequent lowering of the groundwater table. Shoreline sediment excavation (at or just below the water line) is a proven method; however, costs associated with dewatering are relatively high and dewatered fluids would require disposal or treatment prior to discharge into Lake Washington. The technical feasibility of dewatering and dry excavation declines rapidly with increasing excavation depth. Site-specific evaluations estimate that dry excavations cannot be maintained in water depths greater than approximately 12 to 15 feet of water (refer to Appendix D of this FS), and due to this low implementability cofferdam containment was not retained as a representative excavation process option. Upland-based excavation was retained as a representative excavation process option.

C5.5.2 Dredging

Dredging is a method of excavation that allows removal of sediments without the necessary dry conditions required of traditional methods. Dredging is generally accomplished with two main technologies:

- **Hydraulic.** Removal using a cutterhead or auger, which dislodges the sediment, or using plain suction. The dredged material is conveyed along with water using a suction pipe and slurry pumps. The resulting sediment slurry is pumped to a barge or upland location for processing.
- **Mechanical.** Removal using an articulated fixed arm (e.g., backhoe) dredge, enclosed (environmental) bucket, or clamshell bucket on a barge. The mechanical dredge removes the sediment and transfers it into a separate barge for transport to the primary staging area.

Dredging effectiveness may be limited by resuspension, release of COCs (i.e., dissolved, particles, and sheens) to water, volatilization to air during dredging, and residual COCs remaining after dredging (USACE 2008). These effects may be reduced by use of containment (e.g., sheet piles, silt curtains, booms), best management practices (BMPs) (e.g., production rates, bucket control, etc.), and/or by equipment selection.

Mechanical dredging has been used to effectively remove contaminated sediment at many dredging sites. Mechanical dredging can use environmental buckets and operational controls to minimize resuspension. Mechanical dredges are more effective at removing debris than hydraulic dredges. Mechanical dredges are capable of removing most types of small debris without compromising the effectiveness of the dredge to remove sediment. As the size of the debris increases, the effectiveness of the dredge to remove sediment may decrease. Although large debris may cause resuspension, mechanical dredges are still capable of removing the debris (Palermo et al. 2004). Mechanical dredging generally requires handling the dredged material multiple times (e.g., placement on a barge, barge offloading, and transfer to upland staging area).

Hydraulic dredging has also been used successfully to remove contaminated sediments and is advantageous due to the production rate it can achieve under ideal conditions.

Hydraulic dredging is effective for removal of soft sediment, and may cause less resuspension than mechanical sediment removal. In addition, plain suction and specialty hydraulic dredges designed for environmental dredging (e.g., SedVac® by Terra Contracting or the VicVac™ by Brennan) have the potential for greater control of resuspension and releases than navigational hydraulic dredges (USACE 2008). Hydraulic dredges are less effective at handling debris than mechanical dredges and may require debris removal prior to dredging. (Palermo et al. 2004, USACE 2008). Hydraulic dredges can convey the dredged slurry directly to an upland staging area in a pipeline. Because hydraulically dredged sediment has higher water content than mechanical dredging, hydraulically dredged material would require significantly more dewatering than mechanically dredged sediment and would also generate significant amounts of water requiring treatment. Hydraulic dredging would require a greater dewatering and handling area than mechanical dredging.

Real-time positioning systems on both mechanical and hydraulic dredges allow control of position accuracy, inventory control, and real-time tracking.

Both mechanical and hydraulic dredging may be applicable for sediment removal and were retained as representative dredging process options. Containment of dredge areas using sheet piles or silt curtains is also retained for consideration.

C5.6 *Ex Situ* Sediment Treatment Technologies

Potentially applicable treatment technologies for sediment are described and evaluated below.

C5.6.1 Physical Treatment

Physical treatment options include physical separation and solidification/stabilization as discussed below:

Physical Separation. Physical separation is described in Section 3.5.1 above. Excess water can be removed from sediments using process options such as gravity dewatering, filter press, or geotextile tubes, allowing separate treatment and/or disposal of the liquid and solid fractions. Processing may be further performed on the solid fraction to separate coarse- and fine-grained material, as contaminants are generally bound to fine-grained particles and not coarser sands and gravels. Physical separation typically can be accomplished at relatively high to moderate cost and depending on the project may reduce overall treatment/disposal costs by reducing contaminant volume. Therefore, physical separation has been retained as a representative physical treatment process option for sediment.

Solidification/Stabilization. *Ex situ* solidification/stabilization is generally described in Section 3.5.1 above. While stabilization has been successful using relatively coarse sediments and soil, the generally fine-grained nature of Site materials would require the addition of sand and/or gravel to achieve typical structural requirements. Further, the presence of organic materials in Site soils and sediments are of significant concern when applying this process. High organics content can substantially affect stabilization performance and increase costs (which range from $40 to 100/cy; also dependent on water content). Because the stabilization process does not permanently destroy chemical contaminants, the permanence (e.g., long-term durability) of the stabilized material would need to be addressed in bench-scale testing.

Solidification/stabilization as a means of dewatering dredged sediments prior to transport for off-Site disposal is commonly implemented, effective, and relatively low in cost (EPA 2005). Therefore, solidification/stabilization was retained as a potential process option for treating and disposing of dredged sediment.

C5.6.2 Ex Situ Thermal Treatment

Ex situ thermal treatment options included thermal desorption, vitrification, and incinerations as discussed below:

Thermal Desorption. Thermal treatment is described in Section 3.5.2 above. Limitations of thermal desorption for treatment of sediment include high energy requirements for treating wet soils, difficulty in completely treating soils containing high organic content (such as the wood and peaty soils at the Site), and the extensive permitting requirements for on-Site thermal desorption systems. Thermal desorption may be accomplished on Site with a mobile treatment unit or off Site at a permanent treatment facility. Compared to off-Site landfill disposal, thermal desorption is typically more expensive (ranging from $60 to $120/cy), but has the advantage of providing contaminant treatment and destruction rather than containment. Therefore, this process option has been retained for *ex situ* thermal treatment.

Vitrification. Vitrification is described in Section 3.5.2 above. Costs for treating sediment via vitrification are approximately equivalent to those for saturated soil treatment. Therefore, this process option was not retained for this FS.

Incineration. Incineration is described in Section 3.5.2 above. Costs for treating sediment via incineration are approximately equivalent to those for saturated soil treatment. Therefore, this process option was not retained for this FS.

C5.6.3 Ex Situ Chemical/Physical Treatment

Ex situ chemical/physical treatment options include dehalogenation, sediment washing, and solvent extraction as discussed below;

Dehalogenation. Dehalogenation is the process of removing the halogen molecules (e.g., chlorine) from a contaminant in the sediment. In this process, dewatered contaminated sediment is screened, pulverized, and mixed with reagents prior to being heated in a reactor. Reagents used in the process consist of sodium bicarbonate (BCD) or potassium polyethylene glycol (APEG). The dehalogenation process is achieved by either the replacement of the halogen molecules or the decomposition and partial volatilization of the contaminants. The technology targets a relatively small range of contaminants (i.e., PCBs, dioxins, furans, and other halogenated compounds).

Because dehalogenation does not target Site COCs, this process option was not retained for this FS.

Sediment Washing. In sediment washing, sediment is put in contact with an aqueous solution to remove contaminants from the soil particles. The suspension is often also used to separate fine particles from coarser particles, allowing beneficial use of the coarser fraction (if sufficiently clean). The aqueous solution can contain surfactants or other additives to promote contaminant dissolution. Sediment washing is typically more expensive than thermal desorption and has limited effectiveness for removing strongly hydrophobic chemicals such as PAHs, particularly from sediments with a high organic content. Therefore, this process option was not retained.

Solvent Extraction. See Section 3.5.3 above for a description of the solvent extraction process option and its applicability to Site COCs. As discussed, these options were not retained.

C5.6.4 Ex Situ Biological Treatment

See Section 3.5.4 above for a description of biological treatment technology and process options and the applicability to Site COCs. As discussed, these options were not retained.

C5.7 Sediment Disposal Technologies

C5.7.1 On-Site Beneficial Use

Dredged sediments may potentially be beneficially used on the Site if they meet or can be treated to meet applicable cleanup standards. Examples of potential beneficial uses of Site sediments that may be excavated include upland use of wood debris or clean sediments removed as part of habitat restoration or mitigation. Depending on the application (e.g., topsoil or landscaping materials), wood debris dredged for habitat restoration may require amendment through blending (with sand or other granular material) prior to on-Site beneficial use. On-Site beneficial use is the most preferred and likely the least costly method of sediment disposal (ranging between $15 to $30/cy depending on moisture content of the material and whether temporary stockpiling is required). Therefore, on-Site beneficial use has been retained as a technology for this FS.

C5.7.2 On-Site Confined Disposal

Dredged sediments exceeding applicable cleanup standards could potentially be placed on Site in a specially designed upland CDF. Depending on the leachability of confined materials, the CDF could potentially include a liner and a liquid collection system to prevent leachate from contaminating groundwater. On-Site confined disposal can be cheaper than off-Site confined disposal, but requires long-term on-Site management of contaminated materials. Costs for on-Site confined disposal would include those for beneficial use and the cost for developing the facility, which could result in total costs of approximately $35 to $50/cy. This disposal technology has been retained for this FS.

C5.7.3 Off-Site Landfill Disposal

Off-Site landfill disposal process options are described in Section 3.6.3 above. Contaminated Site sediments will likely be characterized as non-hazardous solid wastes and could be shipped via truck and railcar to facilities such as the Klickitat County Landfill in Roosevelt, Washington. This disposal method provides for secure, long-term containment of non-hazardous solid wastes. Costs for dewatering, transport, and disposal may range from approximately $50 to $200/cy. This disposal technology has been retained for this FS.

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APPENDIX D

Remedial Alternatives Cost Estimates

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D1 Introduction

This appendix provides detailed cost estimates for Alternatives 2 through 10. Cost estimates were developed in accordance with EPA cost estimating guidance (*A Guide to Developing and Documenting Cost Estimates During the Feasibility Study*, U.S. Environmental Protection Agency, Office of Emergency and Remedial Response, OSWER Directive 9355.0-75, July 2000) and are FS-level (+50/-30%). Costs are inclusive of contractor overhead and profit. Costs are based on a variety of sources including project experience, vendor and contractor quotes, and available cost databases as noted in each table. Costs are in 2013 dollars. Two total costs were calculated for each alternative: one using Net Present Value (NPV) analysis assuming a discount rate of 1.6 percent, and one with no discount rate for future costs. The discount rate was based on the values published in the Office of Management and Budget (OMB) Circular A-94, Appendix C. For the purposes of these estimates, remedial construction costs were not discounted for alternatives in which construction extends past Year 0. As indicated in Table D-1, these cost estimates range from $33,500,000 (NPV $26,000,000) to $439,000,000 (NPV $409,000,000) for the proposed alternatives.

Table D-1 - Summary of Cost Estimates for EPA-Specified Alternatives

DRAFT FINAL

Quendall Terminals
Renton, Washington

	Total Estimated Cost			
			FS-Level Accuracy Range (with NPV Analysis[2])	
Alternative	Without NPV Analysis	With NPV Analysis[2]	Minus 30%	Plus 50%
Alternative 1 - No Action	$ 0	$ 0	$ 0	$ 0
Alternative 2 - Containment	$ 33,500,000	$ 26,000,000	$ 18,200,000	$ 39,000,000
Alternative 3 - Containment with Targeted PTM Solidification (RR and MC DNAPL Areas) (RR and MC DNAPL Areas)	$ 40,100,000	$ 30,700,000	$ 21,500,000	$ 46,100,000
Alternative 4 - Containment with Targeted PTM Removal (TD, QP-S, and QP-U DNAPL Areas)	$ 49,100,000	$ 44,300,000	$ 31,000,000	$ 66,500,000
Alternative 5 - Containment with Targeted PTM Solidification (RR and MC DNAPL Areas and ≥ 4-Foot-Thickness) and Removal (TD and QP-S DNAPL Areas)	$ 50,700,000	$ 46,500,000	$ 32,600,000	$ 69,800,000
Alternative 6 - Containment with Targeted PTM Solidification (RR and MC DNAPL Areas and ≥ 2-Foot-Thickness) and Removal (TD, QP-S, and QP-U DNAPL Areas)	$ 64,800,000	$ 60,600,000	$ 42,400,000	$ 90,900,000
Alternative 7 - Containment with PTM Solidification (Upland) and Removal (Sediment)	$ 82,800,000	$ 80,400,000	$ 56,300,000	$ 121,000,000
Alternative 8 - Containment with PTM Removal (Upland and Sediment)	$ 142,000,000	$ 140,000,000	$ 98,000,000	$ 210,000,000
Alternative 9 - Containment with Solidification and Removal of Contaminated Soil and Removal of Contaminated Sediment	$ 264,000,000	$ 262,000,000	$ 183,000,000	$ 393,000,000
Alternative 10 - Containment with Removal of Contaminated Soil and Sediment	$ 439,000,000	$ 409,000,000	$ 286,000,000	$ 614,000,000

NPV - Net Present Value

Notes:
1. Estimated costs are rounded to three significant figures.
2. A 1.6% discount rate was used in the net present value analysis.

Table D-2 - Alternative 2 Cost Estimates

DRAFT FINAL

Quendall Terminals
Renton, Washington

Site:	Quendall Terminals		
Remedial Action Description:	Alternative	2	Containment
Cost Estimate Accuracy:	FS Screening Level (+50/-30 percent)		
Key Assumptions and Quantities:	Capping of Upland Soil		
(see Appendix E for calculations)	21.6	acre	total area
	940,896	SF	total area
	133,521	SF	permeable area along shoreline
	14,836	BCY	habitat excavation overlap
	104,544	BCY	total volume based on 3' cap thickness
	Enhanced Natural Recovery - Sand Material		
	14,300	BCY	total volume
	Engineered Sand Cap		
	15,300	BCY	total sand volume
	2,150	BCY	removal volume for offsetting sand cap
	40,000	SF	area for offsetting sand cap
	3.2	acre	DNR lease area
	RCM Reactive Capping materials		
	214,800	SF	area of RCM
	4,100	BCY	total sand volume
	581	BCY	removal volume for offsetting reactive cap
	Amended Sand Capping Materials		
	429	BCY	Bulk Organoclay Material - (PM-199)
	5,727	BCY	Sand
	Soil/Sediment Density		
	1.6	tons/BCY	soil density
	1.3	tons/BCY	sediment density
	0.7	tons/CY	organoclay density
	Volume of sediment removal		
	2,800	BCY	sediment removal
	2,800	BCY	total sediment removal volume (including for offsetting cap)

Item	Quantity	Unit	Unit Cost	Total Cost	Source	Notes
CAPITAL CONSTRUCTION COSTS						
Upland Soil Cap						
Mobilization/Demobilization	1	LS	$ 424,940	$ 424,940	percentage of construction costs	includes temporary facilities for duration of construction
Site Preparation	22	acre	$ 6,900	$ 149,040	Costworks	clearing, grubbing brush and stumps
Geotextile marker layer	104,544	SY	$ 2	$ 158,907	Costworks	non-woven, 120lb tensile strength
Import Fill - Permeable Cap	104,544	BCY	$ 30	$ 3,136,320	project experience	
Compaction	104,544	BCY	$ 5	$ 522,720	project experience	
Habitat Area - excavation	14,836	BCY	$ 6	$ 89,014		
Habitat Area - non-hazardous transport and disposal	23,737	ton	$ 50	$ 1,186,853		
Hydroseeding	14,836	SY	$ 1	$ 8,901	Costworks	includes seed and fertilizer for wetland area
Stormwater collection and detention system	1,500	LF	$ 40	$ 60,000	project experience	media filter drain
Subtotal				*$ 5,736,696*		
Tax	9.5%		$ 5,736,696	$ 544,986		Sales Tax
Contingency	25%		$ 6,281,682	$ 1,570,421		
Total Upland Soil Cap Cost				$ 7,852,103		
Enhanced Natural Recovery						
Mobilization/Demobilization	1	LS	$ 65,664	$ 65,664		
Sand Material	22,880	ton	$ 20	$ 457,600	vendor quote	
Sand Placement	22,880	ton	$ 15	$ 343,200	project experience	ENR placed as one lift
Confirmation of Placement	1	LS	$ 20,000	$ 20,000		
Subtotal				*$ 886,464*		
Tax	9.5%		$ 886,464	$ 84,214		Sales Tax
Contingency	25%		$ 970,678	$ 242,669.52		
Total Enhanced Natural Recovery Cost				$ 1,213,348		
Engineered Sand Cap						
Mobilization/Demobilization	1	LS	$ 81,536	$ 81,536		
Sand Material	24,480	ton	$ 20	$ 489,600	vendor quote	
Sand Placement	24,480	ton	$ 20	$ 489,600	project experience	Sand Cap placed in multiple lifts
Geotextile Separation Layer	40,000	SF	$ 1	$ 20,000	Vendor quote	Only in nearshore area
Confirmation of Placement	1	LS	$ 20,000	$ 20,000		
Subtotal				*$ 1,100,736*		
Tax	9.5%		$ 1,100,736	$ 104,570		Sales Tax
Contingency	25%		$ 1,205,306	$ 301,326		
Total Engineered Sand Cap Cost				$ 1,506,632		
RCM Reactive Capping						
Mobilization/Demobilization	1	LS	$ 99,014	$ 99,014		
Organoclay RCM Material + Transportation	214,800	SF	$ 3	$ 558,480	Quote from Cetco	
Organoclay RCM Placement	214,800	SF	$ 2	$ 429,600	Project experience	
Sand Material	6,560	ton	$ 20	$ 131,200	vendor quote	
Sand Placement	6,560	ton	$ 15	$ 98,400	project experience	Sand over RCM placed in one lift
Confirmation of Placement	1	LS	$ 20,000	$ 20,000		
Subtotal				*$ 1,336,694*		
Tax	9.5%		$ 1,336,694	$ 126,986		Sales Tax
Contingency	25%		$ 1,463,680	$ 365,920		
Total RCM Reactive Capping Cost				$ 1,829,600		
Amended Sand Capping						
Mobilization/Demobilization	1	LS	$ 108,177	$ 108,177		
Bulk Organoclay Material - (PM-199)	300	ton	$ 3,250	$ 975,687	Quote from Cetco	
Sand	9,163	ton	$ 20	$ 183,265	vendor quote	
Material Placement	9,163	ton	$ 20	$ 183,265	project experience	
Confirmation of Placement	1	LS	$ 10,000	$ 10,000		
Subtotal				*$ 1,460,395*		
Tax	9.5%		$ 1,460,395	$ 138,737		Sales Tax
Contingency	25%		$ 1,599,132	$ 399,783		
Total Ameded Sand Capping Cost				$ 1,998,915		
Sediment Removal						
Mobilization/Demobilization	1	LS	$ 32,688	$ 32,688		
Mechanical Dredging	2,800	BCY	$ 35	$ 98,000		Mechanical dredging in nearshore and for offsetting nearshore cap
Transloading/Material Handling	2,800	BCY	$ 15	$ 42,000		
Dewatering	2,800	BCY	$ 10	$ 26,600	vendor quote	Assumes 5% amendment by weight
Water Treatment	1	LS	$ 50,000	$ 50,000	Project experience	
Transportation and Disposal - Non-Hazardous	3,640	ton	$ 50	$ 182,000		Subtitle D landfill disposal
Dredging Confirmation	1	LS	$ 10,000	$ 10,000		
Subtotal				*$ 441,288*		
Tax	9.5%		$ 441,288	$ 41,922		Sales Tax
Contingency	25%		$ 483,210	$ 120,803		
Total Sediment Removal Cost				$ 604,013		
Sediment Environmental Controls and Monitoring						
Water Quality Monitoring	100	day	$ 2,000	$ 200,000		
Water Quality Controls and BMPs (Absorbent Booms, Silt Curtains, Oil Bo	1	LS	$ 25,000	$ 25,000		
Odor Control	10	day	$ 2,500	$ 25,000		
Erosion Protection for Shoreline Area	1	LS	$ 250,000	$ 250,000		
Subtotal				*$ 500,000*		
Tax	9.5%		$ 500,000	$ 47,500		Sales Tax
Contingency	25%		$ 547,500	$ 136,875		
Total Sediment Environmental Controls and Monitoring Cost				$ 684,375		
Subtotal Construction Costs				**$ 15,688,986**		
Professional Services (as percent of construction and contingency costs)						
Project management	5%		$ 15,688,986	$ 784,449		
Remedial design	6%		$ 15,688,986	$ 941,339		Includes treatability studies for remedy components as necessary
Construction management	6%		$ 15,688,986	$ 941,339		
Subtotal				*$ 2,667,128*		
Total Estimated Capital Cost				**$ 18,400,000**		

Table D-2 - Alternative 2 Cost Estimates
Quendall Terminals
Renton, Washington

O&M COSTS								
1st Year O&M								
GW Monitoring	1	LS	$	80,000	$	80,000	Project experience	
Sediment Sand Cap and ENR Sampling	1	LS	$	25,000	$	25,000	Project experience	
Sediment Cap Inspection	1	LS	$	15,000	$	15,000	Project experience	Visual and In-Water (Bathymetric/ Sediment Profile Image)
DNR Lease	3.2	acre	$	20,000	$	64,000		Offshore cap area off property
Subtotal					*$*	*184,000*		
Tax	9.5%		$	184,000	$	17,480		Sales Tax
Contingency	25%		$	201,480	$	50,370		
Total 1st Year O&M Cost					$	251,850		
Annual O&M								
Groundwater Monitoring	1	LS	$	25,000	$	25,000	Project experience	20 wells annually
Upland Cap inspection	6	hour	$	80	$	480	labor estimate	
DNR Lease	3.2	acre	$	20,000	$	64,000		Offshore cap area off property
Subtotal					*$*	*89,480*		
Tax	9.5%		$	89,480	$	8,501		Sales Tax
Contingency	25%		$	97,981	$	24,495		
Total Annual O&M Cost					$	122,476		
Professional Services (as percent of Annual O&M costs)								
Project management/Reporting	10%		$	122,476	$	12,248		
Total, Annual O&M:					**$**	**134,723**		
Total Estimated O&M, 100 Years, No NPV Analysis:					**$**	**13,700,000**		
Periodic Costs								
Reactive Cap								
Replace 25% of RC at 22 yrs					$	300,000		
Replace 25% of RC at 44 yrs					$	300,000		
Replace 25% of RC at 66 yrs					$	300,000		
Replace 25% of RC at 88 yrs					$	300,000		
Sand Cap and ENR								
Sediment Sand Cap and ENR Sampling at 2 years					$	25,000		
Sediment Sand Cap and ENR Sampling at 5 years					$	25,000		
Sediment Sand Cap and ENR Sampling at 10 years					$	25,000		
Sediment Cap Inspection at 2 years					$	15,000		
Sediment Cap Inspection at 5 years					$	15,000		
Sediment Cap Inspection at 10 years					$	15,000		
Sand Cap Shoreline Maintenance at 30 years					$	25,000		
Sand Cap Shoreline Maintenance at 60 years					$	25,000		
Sand Cap Shoreline Maintenance at 90 years					$	25,000		
Subtotal					*$*	*1,395,000*		
TOTAL ESTIMATED COST, NO NPV ANALSYS					**$**	**33,495,000**		
Net Present Value Analysis								
Annual O&M	100	year	$	134,723	$	6,698,529		
1st year O&M	1	LS	$	251,850	$	251,850		
Replace 25% of RC at 22 yrs	1	LS	$	300,000	$	211,573		
Replace 25% of RC at 44 yrs	1	LS	$	300,000	$	149,210		
Replace 25% of RC at 66 yrs	1	LS	$	300,000	$	105,229		
Replace 25% of RC at 88 yrs	1	LS	$	300,000	$	74,212		
Sediment Sand Cap and ENR Sampling at 2 years	1	LS	$	25,000	$	24,219		
Sediment Sand Cap and ENR Sampling at 5 years	1	LS	$	25,000	$	23,093		
Sediment Sand Cap and ENR Sampling at 10 years	1	LS	$	25,000	$	21,331		
Sediment Cap Inspection at 2 years	1	LS	$	15,000	$	14,531		
Sediment Cap Inspection at 5 years	1	LS	$	15,000	$	13,856		
Sediment Cap Inspection at 10 years	1	LS	$	15,000	$	12,798		
Sand Cap Shoreline Maintenance at 30 years	1	LS	$	25,000	$	15,528		
Sand Cap Shoreline Maintenance at 60 years	1	LS	$	25,000	$	9,645		
Sand Cap Shoreline Maintenance at 90 years	1	LS	$	25,000	$	5,991		
2013 discount rate for NPV	1.6%							
Total Estimated O&M and Periodic NPV					**$**	**7,631,596**		
TOTAL ESTIMATED COST					**$**	**26,031,596**		

Notes:
1. Mobilization/Demobilization costs are assumed to include equipment transport and setup, temporary erosion and sedimentation control (TESC) measures, bonds, and insurance.
2. Contingency costs include miscellaneous costs not currently itemized due to the current (preliminary) stage of design development, as well as costs to address unanticipated conditions encountered during construction.

Table D-3 Alternative 3 Cost Estimates

Quendall Terminals
Renton, Washington

Site:	Quendall Terminals		
Remedial Action Description:	Alternative	3	Containment with Targeted PTM Solidification (RR and MC DNAPL Areas)
Cost Estimate Accuracy:	FS Screening Level (+50/-30 percent)		
Key Assumptions and Quantities: (see Appendix E for calculations)	Capping of Upland Soil		
	21.6	acre	total area
	940,896	SF	total area
	133,521	SF	permeable area along shoreline
	14,836	BCY	habitat excavation overlap
	104,544	BCY	total volume based on 3' cap thickness
	Enhanced Natural Recovery - Sand Material		
	14,300	BCY	total volume
	Engineered Sand Cap		
	15,300	BCY	total sand volume
	2,150	BCY	removal volume for offsetting sand cap
	40,000	SF	area for offsetting sand cap
	3.2	acre	DNR lease area
	RCM Reactive Capping materials		
	247,000	SF	area of RCM
	4,700	BCY	total sand volume
	958	BCY	removal volume for offsetting reactive cap
	Soil/Sediment Density		
	1.6	tons/BCY	soil density
	1.3	tons/BCY	sediment density
	Solidification of Upland Source Area Soil		
	17,542	BCY	volume of soil to be solidified
	8,066	BCY	volume of soil at shallow depths to be solidified
	9,476	BCY	volume of deeper soil to be solidified
	Volume of sediment removal		
	3,200	BCY	sediment removal
	3,200	BCY	total sediment removal volume (including for offsetting cap)
	Volumes for DNAPL collection trench installation		
	167	BCY	volume classified as hazardous
	759	BCY	volume classified as non-hazardous
	Volumes for PRB installation		
	367	BCY	volume classified as hazardous
	1,670	BCY	volume classified as non-hazardous
	163	ton	amount of PRB media
	44	BCY	cover material
	820	LF	slurry wall length

Item	Quantity	Unit	Unit Cost	Total Cost	Source	Notes
CAPITAL CONSTRUCTION COSTS						
Upland Soil Cap						
Mobilization/Demobilization	1	LS	$ 424,940	$ 424,940	percentage of construction costs	includes temporary facilities for duration of construction
Site Preparation	22	acre	$ 6,900	$ 149,040	Costworks	clearing, grubbing brush and stumps
Geotextile marker layer	104,544	SY	$ 2	$ 158,907	Costworks	non-woven, 120lb tensile strength
Import Fill - Permeable Cap	104,544	BCY	$ 30	$ 3,136,320	project experience	
Compaction	104,544	BCY	$ 5	$ 522,720	project experience	
Habitat Area - excavation	14,836	BCY	$ 6	$ 89,014		
Habitat Area - non-hazardous transport and disposal	23,737	ton	$ 50	$ 1,186,853		
Hydroseeding	14,836	SY	$ 1	$ 8,901	Costworks	includes seed and fertilizer for wetland area
Stormwater collection and detention system	1,500	LF	$ 40	$ 60,000	project experience	media filter drain
Subtotal				*$ 5,736,696*		
Tax	9.5%		$ 5,736,696	$ 544,986		Sales Tax
Contingency	25%		$ 6,281,682	$ 1,570,421		
Total Upland Soil Cap Cost				$ 7,852,103		
Enhanced Natural Recovery						
Mobilization/Demobilization	1	LS	$ 65,664	$ 65,664		
Sand Material	22,880	ton	$ 20	$ 457,600	vendor quote	
Sand Placement	22,880	ton	$ 15	$ 343,200	project experience	ENR placed as one lift
Confirmation of Placement	1	LS	$ 20,000	$ 20,000		
Subtotal				*$ 886,464*		
Tax	9.5%		$ 886,464	$ 84,214		Sales Tax
Contingency	25%		$ 970,678	$ 242,669.52		
Total Enhanced Natural Recovery Cost				$ 1,213,348		
Engineered Sand Cap						
Mobilization/Demobilization	1	LS	$ 81,536	$ 81,536		
Sand Material	24,480	ton	$ 20	$ 489,600	vendor quote	
Sand Placement	24,480	ton	$ 20	$ 489,600	project experience	Sand Cap placed in multiple lifts
Geotextile Separation Layer	40,000	SF	$ 1	$ 20,000	Vendor quote	Only in nearshore area
Confirmation of Placement	1	LS	$ 20,000	$ 20,000		
Subtotal				*$ 1,100,736*		
Tax	9.5%		$ 1,100,736	$ 104,570		Sales Tax
Contingency	25%		$ 1,205,306	$ 301,326		
Total Engineered Sand Cap Cost				$ 1,506,632		
RCM Reactive Capping						
Mobilization/Demobilization	1	LS	$ 113,552	$ 113,552		
Organoclay RCM Material + Transportation	247,000	SF	$ 3	$ 642,200	Quote from Cetco	
Organoclay RCM Placement	247,000	SF	$ 2	$ 494,000	Project experience	
Sand Material	7,520	ton	$ 20	$ 150,400	vendor quote	
Sand Placement	7,520	ton	$ 15	$ 112,800	project experience	Sand over RCM placed in one lift
Confirmation of Placement	1	LS	$ 20,000	$ 20,000		
Subtotal				*$ 1,532,952*		
Tax	9.5%		$ 1,532,952	$ 145,630		Sales Tax
Contingency	25%		$ 1,678,582	$ 419,646		
Total RCM Reactive Capping Cost				$ 2,098,228		
Upland Soil Solidification						
Mobilization/Demobilization	1	LS	$ 113,395	$ 113,395	percentage of construction costs	includes temporary facilities for duration of construction
Solidification - 8-ft diameter auger	8,066	BCY	$ 70	$ 564,588	project experience	8-ft auger used to cost-effectively treat shallower soils
Solidification - 4-ft diameter auger	9,476	BCY	$ 90	$ 852,847	project experience	4-ft auger used to treat deeper soils, below 8-ft auger limit
Subtotal				*$ 1,530,829*		
Tax	9.5%		$ 1,530,829	$ 145,429		Sales Tax
Contingency	30%		$ 1,676,258	$ 502,877		
Total Upland Soil Solidification Cost				$ 2,179,135		
Sediment Removal						
Mobilization/Demobilization	1	LS	$ 36,672	$ 36,672		
Mechanical Dredging	3,200	BCY	$ 35	$ 112,000		Mechanical dredging in nearshore and for offsetting nearshore cap
Transloading/Material Handling	3,200	BCY	$ 15	$ 48,000		
Dewatering	3,200	BCY	$ 10	$ 30,400	vendor quote	Assumes 5% amendment by weight
Water Treatment	1	LS	$ 50,000	$ 50,000	Project experience	
Transportation and Disposal - Non-Hazardous	4,160	ton	$ 50	$ 208,000		Subtitle D landfill disposal
Dredging Confirmation	1	LS	$ 10,000	$ 10,000		
Subtotal				*$ 495,072*		
Tax	9.5%		$ 495,072	$ 47,032		Sales Tax
Contingency	25%		$ 542,104	$ 135,526		
Total Sediment Removal Cost				$ 677,630		
Sediment Environmental Controls and Monitoring						
Water Quality Monitoring	100	day	$ 2,000	$ 200,000		
Water Quality Controls and BMPs (Absorbent Booms, Silt Curtains, Oil Bo	1	LS	$ 25,000	$ 25,000		
Odor Control	10	day	$ 2,500	$ 25,000		
Noise Monitoring	-	LS	$ -	$ -		
Erosion Protection for Shoreline Area	1	LS	$ 250,000	$ 250,000		
Subtotal				*$ 500,000*		
Tax	9.5%		$ 500,000	$ 47,500		Sales Tax
Contingency	25%		$ 547,500	$ 136,875		
Total Sediment Environmental Controls and Monitoring Cost				$ 684,375		
DNAPL Collection Trenches						
Mobilization/Demobilization	1	LS	$ 51,705	$ 51,705		
Installation	12,500	VSF	$ 40	$ 500,000	Vendor quote	one-pass excavation and backfill including piping and sump
Backfill	1,389	ton	$ 20	$ 27,778	Costworks	pea gravel to 5' bgs, material only
Adsorbent liner	5,000	VSF	$ 4	$ 17,800	Vendor quote	organoclay liner on downgradient wall adjacent PRB - 4 1500ft2 rolls
Transport and Disposal - Non-Hazardous Waste	1,215	ton	$ 50	$ 60,741	project experience	Subtitle D landfill disposal
Transport and Disposal - Hazardous Waste	267	ton	$ 150	$ 40,000	project experience	Subtitle C landfill disposal, assuming no treatment required
Subtotal				*$ 698,024*		
Tax	9.5%		$ 698,024	$ 66,312		Sales Tax
Contingency	25%		$ 764,336	$ 191,084		
Total DNAPL Collection Trenches Cost				$ 955,420		

Table D-3 Alternative 3 Cost Estimates

Quendall Terminals
Renton, Washington

Item	Quantity	Unit	Unit Cost	Cost	Source	Notes
Permeable Treatment Wall						
Mobilization/Demobilization	1	LS	$ 63,731	$ 63,731	Vendor quote	One Pass trencher transport, assembly and disassembly
Excavation and media installation	1	LS	$ 250,000	$ 250,000	Vendor quote	excavate and place GAC
Treatment media	163	ton	$ 920	$ 149,926	Vendor quote	GAC: see Appendix E
Import fill	44	BCY	$ 30	$ 1,333	Project experience	cap for PRB
Monitoring well installation	5	well	$ 4,000	$ 20,000	Project experience	
Transport and Disposal - Non-Hazardous Waste	2,673	ton	$ 50	$ 133,630	project experience	Subtitle D landfill disposal
Transport and Disposal - Hazardous Waste	587	ton	$ 150	$ 88,000	project experience	Subtitle C landfill disposal, assuming no treatment required
Slurry Wall installation	820	LF	$ 188	$ 153,750	Vendor quote	slurry to 25' depth
Subtotal				*$ 860,370*		
Tax	9.5%		$ 860,370	$ 81,735		Sales Tax
Contingency	25%		$ 942,105	$ 235,526		
Total Permeable Treatment Wall Cost				$ 1,177,631		
Subtotal Construction Costs				**$ 18,344,503**		
Professional Services (as percent of construction and contingency costs)						
Project management	5%		$ 18,344,503	$ 917,225		
Remedial design	6%		$ 18,344,503	$ 1,100,670		Includes treatability studies for remedy components as necessary
Construction management	6%		$ 18,344,503	$ 1,100,670		
Subtotal				*$ 3,118,565*		
Total Estimated Capital Cost				**$ 21,500,000**		
O&M COSTS						
1st Year O&M						
GW Monitoring	1	LS	$ 80,000	$ 80,000	Project experience	
Sediment Sand Cap and ENR Sampling	1	LS	$ 25,000	$ 25,000	Project experience	
Sediment Cap Inspection	1	LS	$ 15,000	$ 15,000	Project experience	Visual and In-Water (Bathymetric/ Sediment Profile Image)
DNR Lease	3.2	acre	$ 20,000	$ 64,000		Offshore cap area off property
Subtotal				*$ 184,000*		
Tax	9.5%		$ 184,000	$ 17,480		Sales Tax
Contingency	25%		$ 201,480	$ 50,370		
Total 1st Year O&M Cost				$ 251,850		
Annual O&M						
Groundwater Monitoring	1	LS	$ 25,000	$ 25,000	Project experience	20 wells annually
Upland Cap inspection	6	hour	$ 80	$ 480	labor estimate	
DNR Lease	3.2	acre	$ 20,000	$ 64,000		Offshore cap area off property
Sump Collection and Waste Management	96	hour	$ 80	$ 7,680		monthly
DNAPL Disposal	200	gal	$ 6	$ 1,200		
Subtotal				*$ 98,360*		
Tax	9.5%		$ 98,360	$ 9,344		Sales Tax
Contingency	25%		$ 107,704	$ 26,926		
Total Annual O&M Cost				$ 134,630		
Professional Services (as percent of Annual O&M costs)						
Project management/Reporting	10%		$ 134,630	$ 13,463		
Total, Annual O&M:				**$ 148,093**		
Total Estimated O&M, 100 Years, No NPV Analysis:				**$ 15,100,000**		
Periodic Costs						
Reactive Cap						
Replace 25% of RC at 22 yrs				$ 300,000		
Replace 25% of RC at 44 yrs				$ 300,000		
Replace 25% of RC at 66 yrs				$ 300,000		
Replace 25% of RC at 88 yrs				$ 300,000		
Sand Cap and ENR						
Sediment Sand Cap and ENR Sampling at 2 years				$ 25,000		
Sediment Sand Cap and ENR Sampling at 5 years				$ 25,000		
Sediment Sand Cap and ENR Sampling at 10 years				$ 25,000		
Sediment Cap Inspection at 2 years				$ 15,000		
Sediment Cap Inspection at 5 years				$ 15,000		
Sediment Cap Inspection at 10 years				$ 15,000		
Sand Cap Shoreline Maintenance at 30 years				$ 25,000		
Sand Cap Shoreline Maintenance at 60 years				$ 25,000		
Sand Cap Shoreline Maintenance at 90 years				$ 25,000		
Permeable treatment wall						
Replace Media at 22 yrs				$ 528,842		includes mob/demob, excavation, media, and $400 per ton disposal fee
Replace Media at 44 yrs				$ 528,842		includes mob/demob, excavation, media, and $400 per ton disposal fee
Replace Media at 66 yrs				$ 528,842		includes mob/demob, excavation, media, and $400 per ton disposal fee
Replace Media at 88 yrs				$ 528,842		includes mob/demob, excavation, media, and $400 per ton disposal fee
Subtotal				*$ 3,510,369*		
TOTAL ESTIMATED COST, NO NPV ANALSYS				**$ 40,110,369**		
Net Present Value Analysis						
Annual O&M	100	year	$ 148,093	$ 7,363,292		
1st year O&M	1	LS	$ 251,850	$ 251,850		
Replace 25% of RC at 22 yrs	1	LS	$ 300,000	$ 211,573		
Replace 25% of RC at 44 yrs	1	LS	$ 300,000	$ 149,210		
Replace 25% of RC at 66 yrs	1	LS	$ 300,000	$ 105,229		
Replace 25% of RC at 88 yrs	1	LS	$ 300,000	$ 74,212		
Sediment Sand Cap and ENR Sampling at 2 years	1	LS	$ 25,000	$ 24,219		
Sediment Sand Cap and ENR Sampling at 5 years	1	LS	$ 25,000	$ 23,093		
Sediment Sand Cap and ENR Sampling at 10 years	1	LS	$ 25,000	$ 21,331		
Sediment Cap Inspection at 2 years	1	LS	$ 15,000	$ 14,531		
Sediment Cap Inspection at 5 years	1	LS	$ 15,000	$ 13,856		
Sediment Cap Inspection at 10 years	1	LS	$ 15,000	$ 12,798		
Sand Cap Shoreline Maintenance at 30 years	1	LS	$ 25,000	$ 15,528		
Sand Cap Shoreline Maintenance at 60 years	1	LS	$ 25,000	$ 9,645		
Sand Cap Shoreline Maintenance at 90 years	1	LS	$ 25,000	$ 5,991		
Replace PRB Media at 22 yrs	1	LS	$ 528,842	$ 372,962		
Replace PRB Media at 44 yrs	1	LS	$ 528,842	$ 263,029		
Replace PRB Media at 66 yrs	1	LS	$ 528,842	$ 185,499		
Replace PRB Media at 88 yrs	1	LS	$ 528,842	$ 130,822		
2013 discount rate for NPV	1.6%					
Total Estimated O&M and Periodic NPV				**$ 9,248,669**		
TOTAL ESTIMATED COST				**$ 30,748,669**		

Notes:
1. Mobilization/Demobilization costs are assumed to include equipment transport and setup, temporary erosion and sedimentation control (TESC) measures, bonds, and insurance.
2. Contingency costs include miscellaneous costs not currently itemized due to the current (preliminary) stage of design development, as well as costs to address unanticipated conditions encountered during construction.

Table D-4 - Alternative 4 Cost Estimate

Quendall Terminals
Renton, Washington

Site:	Quendall Terminals		
Remedial Action Description:	Alternative	4	Containment with Targeted PTM Removal (TD, QP-S, and QP-U DNAPL Areas)
Cost Estimate Accuracy:	FS Screening Level (+50/-30 percent)		
Key Assumptions and Quantities:	Capping of Upland Soil		
(see Appendix E for calculations)	21.6	acre	total area
	940,896	SF	total area
	133,521	SF	permeable area along shoreline
	12,441	BCY	habitat excavation overlap
	104,544	BCY	total volume based on 3' cap thickness
	Enhanced Natural Recovery - Sand Material		
	14,300	BCY	total volume
	Engineered Sand Cap		
	15,800	BCY	total sand volume
	2,150	BCY	removal volume for offsetting sand cap
	40,000	SF	area for offsetting sand cap
	0.5	acre	DNR lease area
	RCM Reactive Capping materials		
	85,600	SF	area of RCM
	1,700	BCY	total sand volume
	570	BCY	removal volume for offsetting reactive cap
	Soil/Sediment Density		
	1.6	tons/BCY	soil density
	1.3	tons/BCY	sediment density
	0.7	tons/CY	organoclay density
	Removal of Upland Source Area Soil		
	12,700	BCY	total volume
	0.5	acre	total area
	2,286	BCY	volume classified as hazardous
	10,414	BCY	volume classified as non-hazardous
	Volume of sediment removal		
	23,200	BCY	sediment removal
	25,900	BCY	total sediment removal volume (including for offsetting cap)
	11,000	BCY	mechanical dredging
	12,200	BCY	hydraulic dredging
	510	BCY	residual cover - organoclay
	2,300	BCY	residual cover - sand
	20,400	BCY	backfill
	35,000	SF	sheet pile area
	Volumes for DNAPL collection trench installation		
	167	BCY	volume classified as hazardous
	759	BCY	volume classified as non-hazardous
	Volumes for PRB installation		
	367	BCY	volume classified as hazardous
	1,670	BCY	volume classified as non-hazardous
	163	ton	amount of PRB media
	44	BCY	cover material
	820	LF	slurry wall length
	Dewatering to maintain wet removal for upland soil		
	120	gpm	maximum upland dewatering rate
	120	gpm	average upland dewatering rate
	6	each	deep aquifer depressurization wells
	0.12	year	upland soil removal time
	16	feet	average excavation depth
	35	feet	min.embed. depth
	10,109	SF	shoring wall area

Item	Quantity	Unit	Unit Cost	Total Cost	Source	Notes
CAPITAL CONSTRUCTION COSTS						
Upland Soil Cap						
Mobilization/Demobilization	1	LS	$ 357,404	$ 357,404	percentage of construction costs	includes temporary facilities for duration of construction
Site Preparation	22	acre	$ 6,900	$ 149,040	Costworks	clearing, grubbing brush and stumps
Geotextile marker layer	104,544	SY	$ 2	$ 158,907	Costworks	non-woven, 120lb tensile strength
Import Fill - Permeable Cap	104,544	BCY	$ 30	$ 3,136,320	project experience	
Compaction	104,544	BCY	$ 5	$ 522,720	project experience	
Habitat Area - excavation	12,441	BCY	$ 6	$ 74,643		
Habitat Area - non-hazardous transport and disposal	19,905	ton	$ 50	$ 995,244		
Hydroseeding	14,836	SY	$ 1	$ 8,901	Costworks	includes seed and fertilizer for wetland area
Stormwater collection and detention system	1,500	LF	$ 40	$ 60,000	project experience	media filter drain
Subtotal				*$ 5,463,180*		
Tax	9.5%		$ 5,463,180	$ 519,002		Sales Tax
Contingency	25%		$ 5,982,183	$ 1,495,546		
Total Upland Soil Cap Cost				$ 7,477,728		
Enhanced Natural Recovery						
Mobilization/Demobilization	1	LS	$ 57,456	$ 57,456		
Sand Material	22,880	ton	$ 20	$ 457,600	vendor quote	
Sand Placement	22,880	ton	$ 15	$ 343,200	project experience	ENR placed as one lift
Confirmation of Placement	1	LS	$ 20,000	$ 20,000		
Subtotal				*$ 878,256*		
Tax	9.5%		$ 878,256	$ 83,434		Sales Tax
Contingency	25%		$ 961,690	$ 240,422.58		
Total Enhanced Natural Recovery Cost				$ 1,202,113		
Engineered Sand Cap						
Mobilization/Demobilization	1	LS	$ 73,584	$ 73,584		
Sand Material	25,280	ton	$ 20	$ 505,600	vendor quote	
Sand Placement	25,280	ton	$ 20	$ 505,600	project experience	Sand Cap placed in multiple lifts
Geotextile Separation Layer	40,000	SF	$ 1	$ 20,000	Vendor quote	Only in nearshore area
Confirmation of Placement	1	LS	$ 20,000	$ 20,000		
Subtotal				*$ 1,124,784*		
Tax	9.5%		$ 1,124,784	$ 106,854		Sales Tax
Contingency	25%		$ 1,231,638	$ 307,910		
Total Engineered Sand Cap Cost				$ 1,539,548		
RCM Reactive Capping						
Mobilization/Demobilization	1	LS	$ 35,627	$ 35,627		
Organoclay RCM Material + Transportation	85,600	SF	$ 3	$ 222,560	Quote from Cetco	
Organoclay RCM Placement	85,600	SF	$ 2	$ 171,200	Project experience	
Sand Material	2,720	ton	$ 20	$ 54,400	vendor quote	
Sand Placement	2,720	ton	$ 15	$ 40,800	project experience	Sand over RCM placed in one lift
Confirmation of Placement	1	LS	$ 20,000	$ 20,000		
Subtotal				*$ 544,587*		
Tax	9.5%		$ 544,587	$ 51,736		Sales Tax
Contingency	25%		$ 596,323	$ 149,081		
Total RCM Reactive Capping Cost				$ 745,404		
Upland Soil Removal						
Mobilization/Demobilization	1	LS	$ 245,881	$ 245,881	percentage of construction costs	includes temporary facilities for duration of construction
Excavation	12,700	BCY	$ 6	$ 76,200	project experience	
Import Fill	12,700	BCY	$ 30	$ 381,000	project experience	
Soil Handling and Stockpiling	12,700	BCY	$ 5	$ 63,500	project experience	segregation into hazardous/non-hazardous
Analytical Sampling	200	ea	$ 500	$ 100,000	project experience	VOCs and SVOCs
Compaction	12,700	BCY	$ 5	$ 63,500	project experience	
Transport and Disposal - Non-Hazardous Waste	16,662	ton	$ 50	$ 833,120	project experience	Subtitle D landfill disposal
Transport and Disposal - Hazardous Waste	3,658	ton	$ 150	$ 548,640	project experience	Subtitle C landfill disposal, assuming no treatment required
Shoring	10,109	SF	$ 92	$ 930,055	project experience	sheet pile - stiffened to allow excavation in the wet (see Appendix F)
Dewatering - Deep Aquifer Depressurization Wells and Pumps	6	ea	$ 40,000	$ 240,000	project experience	
Dewatering - Equalization Tank	2	month	$ 980	$ 1,960	project experience	Rental - 20,000 gallon tank
Dewatering - Treatment system	2	month	$ 8,066	$ 16,132	Vendor quote	rental system: DNAPL separation, air stripping, filtration, GAC vessels
Dewatering - Carbon Replacement	45	day	$ 72	$ 3,198	Vendor quote	based on usage rate of 65 lb/day @ 50gpm - $0.46/lb
Dewatering - Carbon Disposal	3	ton	$ 400	$ 1,391	Vendor quote	
Dewatering - Coagulant	64	lb	$ 2	$ 145	Vendor quote	$2.25 per lb, 1mg/L concentration, average flow rate
Dewatering - Miscellaneous Equipment	20%		$ 363,804	$ 72,761	percentage of dewatering capital cos	
Dewatering - Equipment Operation and Maintenance	45	day	$ 700	$ 31,200	labor estimate	1 full-time operator, $70/hr, 10hr/day
Dewatering - Discharge Fee	7,702,062	gal	$ 0	$ 64,697	project experience	$0.0084/gal discharge rate for city of Renton sewer at adjacent site
Dewatering - Power	2	month	$ 2,540	$ 5,080	project experience	$0.0996/KWH estimated power rate
Monitoring Well Installation	20	ea	$ 4,000	$ 80,000	project experience	confirmation monitoring program
Subtotal				*$ 3,758,460*		
Tax	9.5%		$ 3,758,460	$ 357,054		Sales Tax
Contingency	35%		$ 4,115,514	$ 1,440,430		
Total Upland Soil Removal Cost				$ 5,555,943		

Table D-4 - Alternative 4 Cost Estimate
Quendall Terminals
Renton, Washington

Item	Quantity	Unit	Unit Cost	Cost	Source	Notes
Sediment Removal						
Mobilization/Demobilization	1	LS	$ 439,800	$ 439,800		
Mechanical Dredging	13,720	BCY	$ 35	$ 480,194		Mechanical dredging in nearshore and for offsetting nearshore cap
Hydraulic Dredging	12,200	BCY	$ 60	$ 732,000	Project experience	Assumes specialty hydraulic for T-Dock/Offshore
Debris Removal and Disposal	1	LS	$ 50,000	$ 50,000		Removal of piling
Transloading/Material Handling	25,900	BCY	$ 15	$ 388,500		
Dewatering	25,900	BCY	$ 10	$ 246,050	vendor quote	Assumes 5% amendment by weight
Water Treatment	1	LS	$ 200,000	$ 200,000	Project experience	
Residuals Cover Bulk Organoclay Material - (PM-199)	365	ton	$ 3,250	$ 1,185,941	Quote from Cetco	
Residuals Cover Sand Material	3,680	ton	$ 20	$ 73,600	vendor quote	
Residuals Cover Material Placement	4,045	ton	$ 15	$ 60,674	project experience	
Backfill Material	32,640	ton	$ 20	$ 652,800	vendor quote	
Backfill Material Placement	32,640	ton	$ 15	$ 489,600	project experience	Backfill placed in bulk
Transportation and Disposal - Non-Hazardous	33,670	ton	$ 50	$ 1,683,500		Subtitle D landfill disposal
Dredging Confirmation	1	LS	$ 40,000	$ 40,000		
Subtotal				*$ 6,722,659*		
Tax	9.5%		$ 6,722,659	$ 638,653		Sales Tax
Contingency	25%		$ 7,361,312	$ 1,840,328		
Total Sediment Removal Cost				$ 9,201,640		
Sheet Pile Enclosure						
Mobilization/Demobilization	1	LS	$ 220,500	$ 220,500	Project experience	
Steel Unit Cost	35,000	SF	$ 35	$ 1,225,000	Project experience	
Installation Unit Cost	35,000	SF	$ 45	$ 1,575,000	Project experience	
Removal Unit Cost	35,000	SF	$ 15	$ 525,000	Project experience	
Salvage Unit Value	1,750,000	lb	$ (0.1)	$ (175,000)	Project experience	50 pounds per sf
Subtotal				*$ 3,370,500*		
Tax	9.5%		$ 3,370,500	$ 320,198		Sales Tax
Contingency	25%		$ 3,690,698	$ 922,674		
Total Sheet Pile Enclosure Cost				$ 4,613,372		
Sediment Environmental Controls and Monitoring						
Water Quality Monitoring	175	day	$ 2,500	$ 437,500		
Water Quality Controls and BMPs (Absorbent Booms, Silt Curtains, Oil Boom	1	LS	$ 75,000	$ 75,000		
Odor Control	60	day	$ 2,500	$ 150,000		
Noise Monitoring	1	LS	$ 15,000	$ 15,000		
Erosion Protection for Shoreline Area	1	LS	$ 250,000	$ 250,000		
Subtotal				*$ 927,500*		
Tax	9.5%		$ 927,500	$ 88,113		Sales Tax
Contingency	25%		$ 1,015,613	$ 253,903		
Total Sediment Environmental Controls and Monitoring Cost				$ 1,269,516		
DNAPL Collection Trenches						
Mobilization/Demobilization	1	LS	$ 45,242	$ 45,242		
Installation	12,500	VSF	$ 40	$ 500,000	Vendor quote	one-pass excavation and backfill including piping and sump
Backfill	1,389	ton	$ 20	$ 27,778	Costworks	pea gravel to 5' bgs, material only
Adsorbent liner	5,000	VSF	$ 4	$ 17,800	Vendor quote	organoclay liner on downgradient wall adjacent PRB - 4 1500ft2 rolls
Transport and Disposal - Non-Hazardous Waste	1,215	ton	$ 50	$ 60,741	project experience	Subtitle D landfill disposal
Transport and Disposal - Hazardous Waste	267	ton	$ 150	$ 40,000	project experience	Subtitle C landfill disposal, assuming no treatment required
Subtotal				*$ 691,561*		
Tax	9.5%		$ 691,561	$ 65,698		Sales Tax
Contingency	25%		$ 757,259	$ 189,315		
Total DNAPL Collection Trenches Cost				$ 946,574		
Permeable Treatment Wall						
Mobilization/Demobilization	1	LS	$ 55,765	$ 55,765	Vendor quote	One Pass trencher transport, assembly and disassembly
Excavation and media installation	1	LS	$ 250,000	$ 250,000	Vendor quote	excavate and place GAC
Treatment media	163	ton	$ 920	$ 149,926	Vendor quote	GAC: see Appendix E
Import fill	44	BCY	$ 30	$ 1,333	Project experience	cap for PRB
Monitoring well installation	5	well	$ 4,000	$ 20,000	Project experience	
Transport and Disposal - Non-Hazardous Waste	2,673	ton	$ 50	$ 133,630	project experience	Subtitle D landfill disposal
Transport and Disposal - Hazardous Waste	587	ton	$ 150	$ 88,000	project experience	Subtitle C landfill disposal, assuming no treatment required
Slurry Wall installation	820	LF	$ 188	$ 153,750	Vendor quote	slurry to 25' depth
Subtotal				*$ 852,404*		
Tax	9.5%		$ 852,404	$ 80,978		Sales Tax
Contingency	25%		$ 933,382	$ 233,345		
Total Permeable Treatment Wall Cost				$ 1,166,727		
Subtotal Construction Costs				**$ 33,718,565**		
Professional Services (as percent of construction and contingency costs)						
Project management	5%		$ 33,718,565	$ 1,685,928		
Remedial design	6%		$ 33,718,565	$ 2,023,114		Includes treatability studies for remedy components as necessary
Construction management	6%		$ 33,718,565	$ 2,023,114		
Subtotal				*$ 5,732,156*		
Total Estimated Capital Cost				**$ 39,500,000**		
O&M COSTS						
1st Year O&M						
GW Monitoring	1	LS	$ 80,000	$ 80,000	Project experience	
Sediment Sand Cap and ENR Sampling	1	LS	$ 25,000	$ 25,000	Project experience	
Sediment Cap Inspection	1	LS	$ 15,000	$ 15,000	Project experience	Visual and In-Water (Bathymetric/ Sediment Profile Image)
Backfilled Area Surface Sediment Monitoring	1	LS	$ 25,000	$ 25,000		
DNR Lease	0.5	acre	$ 20,000	$ 10,000		Offshore cap area off property
Subtotal				*$ 155,000*		
Tax	9.5%		$ 155,000	$ 14,725		Sales Tax
Contingency	25%		$ 169,725	$ 42,431		
Total 1st Year O&M Cost				$ 212,156		
Annual O&M						
Groundwater Monitoring	1	LS	$ 25,000	$ 25,000	Project experience	20 wells annually
Upland Cap inspection	6	hour	$ 80	$ 480	labor estimate	
DNR Lease	0.5	acre	$ 20,000	$ 10,000		Offshore cap area off property
Sump Collection and Waste Management	96	hour	$ 80	$ 7,680		monthly
DNAPL Disposal	200	gal	$ 6	$ 1,200		
Subtotal				*$ 44,360*		
Tax	9.5%		$ 44,360	$ 4,214		Sales Tax
Contingency	25%		$ 48,574	$ 12,144		
Total Annual O&M Cost				$ 60,718		
Professional Services (as percent of Annual O&M costs)						
Project management/Reporting	10%		$ 60,718	$ 6,072		
Total, Annual O&M:				**$ 66,790**		
Total Estimated O&M, 100 Years, No NPV Analysis:				**$ 6,900,000**		
Periodic Costs						
Reactive Cap						
Replace 25% of RC at 22 yrs				$ 110,000		
Replace 25% of RC at 44 yrs				$ 110,000		
Replace 25% of RC at 66 yrs				$ 110,000		
Replace 25% of RC at 88 yrs				$ 110,000		
Sand Cap and ENR						
Sediment Sand Cap and ENR Sampling at 2 years				$ 25,000		
Sediment Sand Cap and ENR Sampling at 5 years				$ 25,000		
Sediment Sand Cap and ENR Sampling at 10 years				$ 25,000		
Sediment Cap Inspection at 2 years				$ 15,000		
Sediment Cap Inspection at 5 years				$ 15,000		
Sediment Cap Inspection at 10 years				$ 15,000		
Sand Cap Shoreline Maintenance at 30 years				$ 25,000		
Sand Cap Shoreline Maintenance at 60 years				$ 25,000		
Sand Cap Shoreline Maintenance at 90 years				$ 25,000		
Permeable treatment wall						
Replace Media at 22 yrs				$ 520,876		includes mob/demob, excavation, media, and $400 per ton disposal fee
Replace Media at 44 yrs				$ 520,876		includes mob/demob, excavation, media, and $400 per ton disposal fee
Replace Media at 66 yrs				$ 520,876		includes mob/demob, excavation, media, and $400 per ton disposal fee
Replace Media at 88 yrs				$ 520,876		includes mob/demob, excavation, media, and $400 per ton disposal fee
Subtotal				*$ 2,718,503*		
TOTAL ESTIMATED COST, NO NPV ANALSYS				**$ 49,118,503**		
Net Present Value Analysis						
Annual O&M	100	year	$ 66,790	$ 3,320,818		
1st year O&M	1	LS	$ 212,156	$ 212,156		
Replace 25% of RC at 22 yrs	1	LS	$ 110,000	$ 77,577		
Replace 25% of RC at 44 yrs	1	LS	$ 110,000	$ 54,710		
Replace 25% of RC at 66 yrs	1	LS	$ 110,000	$ 38,584		
Replace 25% of RC at 88 yrs	1	LS	$ 110,000	$ 27,211		
Sediment Sand Cap and ENR Sampling at 2 years	1	LS	$ 25,000	$ 24,219		
Sediment Sand Cap and ENR Sampling at 5 years	1	LS	$ 25,000	$ 23,093		
Sediment Sand Cap and ENR Sampling at 10 years	1	LS	$ 25,000	$ 21,331		
Sediment Cap Inspection at 2 years	1	LS	$ 15,000	$ 14,531		
Sediment Cap Inspection at 5 years	1	LS	$ 15,000	$ 13,856		
Sediment Cap Inspection at 10 years	1	LS	$ 15,000	$ 12,798		
Sand Cap Shoreline Maintenance at 30 years	1	LS	$ 25,000	$ 15,528		
Sand Cap Shoreline Maintenance at 60 years	1	LS	$ 25,000	$ 9,645		
Sand Cap Shoreline Maintenance at 90 years	1	LS	$ 25,000	$ 5,991		
Replace PRB Media at 22 yrs	1	LS	$ 520,876	$ 367,344		
Replace PRB Media at 44 yrs	1	LS	$ 520,876	$ 259,066		
Replace PRB Media at 66 yrs	1	LS	$ 520,876	$ 182,705		
Replace PRB Media at 88 yrs	1	LS	$ 520,876	$ 128,851		
2013 discount rate for NPV	1.6%					
Total Estimated O&M and Periodic NPV				**$ 4,810,014**		
TOTAL ESTIMATED COST				**$ 44,310,014**		

Notes:
1. Mobilization/Demobilization costs are assumed to include equipment transport and setup, temporary erosion and sedimentation control (TESC) measures, bonds, and insurance.
2. Contingency costs include miscellaneous costs not currently itemized due to the current (preliminary) stage of design development, as well as costs to address unanticipated conditions encountered during construction.

Table D-5 - Alternative 5 Cost Estimate
Quendall Terminals
Renton, Washington

Site:	Quendall Terminals	
Remedial Action Description:	Alternative 5	Containment with Targeted PTM Solidfication (RR, MC, and QP-U DNAPL Areas and ≥ 4-Foot-Thickness) and Removal (TD and QP-S DNAPL Areas)
Cost Estimate Accuracy:	FS Screening Level (+50/-30 percent)	

Key Assumptions and Quantities: (see Appendix E for calculations)	Quantity	Unit	Description
Capping of Upland Soil			
	21.6	acre	total area
	940,896	SF	total area
	133,521	SF	permeable area along shoreline
	14,836	BCY	habitat excavation overlap
	104,544	BCY	total volume based on 3' cap thickness
Enhanced Natural Recovery - Sand Material			
	14,300	BCY	total volume
Engineered Sand Cap			
	15,800	BCY	total sand volume
	2,150	BCY	removal volume for offsetting sand cap
	40,000	SF	area for offsetting sand cap
	0.5	acre	DNR lease area
RCM Reactive Capping materials			
	85,600	SF	area of RCM
	1,700	BCY	total sand volume
	570	BCY	removal volume for offsetting reactive cap
Soil/Sediment Density			
	1.6	tons/BCY	soil density
	1.3	tons/BCY	sediment density
	0.7	tons/CY	organoclay density
Solidification of Upland Source Area Soil			
	78,913	BCY	volume of soil to be solidified
	69,437	BCY	volume of soil at shallow depths to be solidified
	9,476	BCY	volume of deeper soil to be solidified
Volume of sediment removal			
	23,200	BCY	sediment removal
	25,900	BCY	total sediment removal volume (including for offsetting cap)
	11,000	BCY	mechanical dredging
	12,200	BCY	hydraulic dredging
	510	BCY	residual cover - organoclay
	2,300	BCY	residual cover - sand
	20,400	BCY	backfill
	35,000	SF	sheet pile area
Volumes for PRB installation			
	367	BCY	volume classified as hazardous
	1,670	BCY	volume classified as non-hazardous
	163	ton	amount of PRB media
	44	BCY	cover material
	820	LF	slurry wall length

Item	Quantity	Unit	Unit Cost	Total Cost	Source	Notes
CAPITAL CONSTRUCTION COSTS						
Upland Soil Cap						
Mobilization/Demobilization	1	LS	$ 371,823	$ 371,823	percentage of construction costs	includes temporary facilities for duration of construction
Site Preparation	22	acre	$ 6,900	$ 149,040	Costworks	clearing, grubbing brush and stumps
Geotextile marker layer	104,544	SY	$ 2	$ 158,907	Costworks	non-woven, 120lb tensile strength
Import Fill - Permeable Cap	104,544	BCY	$ 30	$ 3,136,320	project experience	
Compaction	104,544	BCY	$ 5	$ 522,720	project experience	
Habitat Area - excavation	14,836	BCY	$ 6	$ 89,014		
Habitat Area - non-hazardous transport and disposal	23,737	ton	$ 50	$ 1,186,853		
Hydroseeding	14,836	SY	$ 1	$ 8,901	Costworks	includes seed and fertilizer for wetland area
Stormwater collection and detention system	1,500	LF	$ 40	$ 60,000	project experience	media filter drain
Subtotal				*$ 5,683,579*		
Tax	9.5%		$ 5,683,579	$ 539,940		Sales Tax
Contingency	25%		$ 6,223,518	$ 1,555,880		
Total Upland Soil Cap Cost				$ 7,779,398		
Enhanced Natural Recovery						
Mobilization/Demobilization	1	LS	$ 57,456	$ 57,456		
Sand Material	22,880	ton	$ 20	$ 457,600	vendor quote	
Sand Placement	22,880	ton	$ 15	$ 343,200	project experience	ENR placed as one lift
Confirmation of Placement	1	LS	$ 20,000	$ 20,000		
Subtotal				*$ 878,256*		
Tax	9.5%		$ 878,256	$ 83,434		Sales Tax
Contingency	25%		$ 961,690	$ 240,422.58		
Total Enhanced Natural Recovery Cost				$ 1,202,113		
Engineered Sand Cap						
Mobilization/Demobilization	1	LS	$ 73,584	$ 73,584		
Sand Material	25,280	ton	$ 20	$ 505,600	vendor quote	
Sand Placement	25,280	ton	$ 20	$ 505,600	project experience	Sand Cap placed in multiple lifts
Geotextile Separation Layer	40,000	SF	$ 1	$ 20,000	Vendor quote	Only in nearshore area
Confirmation of Placement	1	LS	$ 20,000	$ 20,000		
Subtotal				*$ 1,124,784*		
Tax	9.5%		$ 1,124,784	$ 106,854		Sales Tax
Contingency	25%		$ 1,231,638	$ 307,910		
Total Engineered Sand Cap Cost				$ 1,539,548		
RCM Reactive Capping						
Mobilization/Demobilization	1	LS	$ 35,627	$ 35,627		
Organoclay RCM Material + Transportation	85,600	SF	$ 3	$ 222,560	Quote from Cetco	
Organoclay RCM Placement	85,600	SF	$ 2	$ 171,200	Project experience	
Sand Material	2,720	ton	$ 20	$ 54,400	vendor quote	
Sand Placement	2,720	ton	$ 15	$ 40,800	project experience	Sand over RCM placed in one lift
Confirmation of Placement	1	LS	$ 20,000	$ 20,000		
Subtotal				*$ 544,587*		
Tax	9.5%		$ 544,587	$ 51,736		Sales Tax
Contingency	25%		$ 596,323	$ 149,081		
Total RCM Reactive Capping Cost				$ 745,404		
Upland Soil Solidification						
Mobilization/Demobilization	1	LS	$ 399,939	$ 399,939	percentage of construction costs	includes temporary facilities for duration of construction
Solidification - 8-ft diameter auger	69,437	BCY	$ 70	$ 4,860,566	project experience	8-ft auger used to cost-effectively treat shallower soils
Solidification - 4-ft diameter auger	9,476	BCY	$ 90	$ 852,847	project experience	4-ft auger used to treat deeper soils, below 8-ft auger limit
Subtotal				*$ 6,113,352*		
Tax	9.5%		$ 6,113,352	$ 580,768		Sales Tax
Contingency	30%		$ 6,694,120	$ 2,008,236		
Total Upland Soil Solidification Cost				$ 8,702,356		
Sediment Removal						
Mobilization/Demobilization	1	LS	$ 439,800	$ 439,800		
Mechanical Dredging	13,720	BCY	$ 35	$ 480,194		Mechanical dredging in nearshore and for offsetting nearshore cap
Hydraulic Dredging	12,200	BCY	$ 60	$ 732,000	Project experience	Assumes specialty hydraulic for T-Dock/Offshore
Debris Removal and Disposal	1	LS	$ 50,000	$ 50,000		Removal of piling
Transloading/Material Handling	25,900	BCY	$ 15	$ 388,500		
Dewatering	25,900	BCY	$ 10	$ 246,050	vendor quote	Assumes 5% amendment by weight
Water Treatment	1	LS	$ 200,000	$ 200,000	Project experience	
Residuals Cover Bulk Organoclay Material - (PM-199)	365	ton	$ 3,250	$ 1,185,941	Quote from Cetco	
Residuals Cover Sand Material	3,680	ton	$ 20	$ 73,600	vendor quote	
Residuals Cover Material Placement	4,045	ton	$ 15	$ 60,674	project experience	
Backfill Material	32,640	ton	$ 20	$ 652,800	vendor quote	
Backfill Material Placement	32,640	ton	$ 15	$ 489,600	project experience	Backfill placed in bulk
Transportation and Disposal - Non-Hazardous	33,670	ton	$ 50	$ 1,683,500		Subtitle D landfill disposal
Dredging Confirmation	1	LS	$ 40,000	$ 40,000		
Subtotal				*$ 6,722,659*		
Tax	9.5%		$ 6,722,659	$ 638,653		Sales Tax
Contingency	25%		$ 7,361,312	$ 1,840,328		
Total Sediment Removal Cost				$ 9,201,640		
Sheet Pile Enclosure						
Mobilization/Demobilization	1	LS	$ 220,500	$ 220,500	Project experience	
Steel Unit Cost	35,000	SF	$ 35	$ 1,225,000	Project experience	
Installation Unit Cost	35,000	SF	$ 45	$ 1,575,000	Project experience	
Removal Unit Cost	35,000	SF	$ 15	$ 525,000	Project experience	
Salvage Unit Value	1,750,000	lb	$ (0.1)	$ (175,000)	Project experience	50 pounds per sf
Subtotal				*$ 3,370,500*		
Tax	9.5%		$ 3,370,500	$ 320,198		Sales Tax
Contingency	25%		$ 3,690,698	$ 922,674		
Total Sheet Pile Enclosure Cost				$ 4,613,372		
Sediment Environmental Controls and Monitoring						
Water Quality Monitoring	175	day	$ 2,500	$ 437,500		
Water Quality Controls and BMPs (Absorbent Booms, Silt Curtains, Oil Bo	1	LS	$ 75,000	$ 75,000		
Odor Control	60	day	$ 2,500	$ 150,000		
Noise Monitoring	1	LS	$ 15,000	$ 15,000		
Erosion Protection for Shoreline Area	1	LS	$ 250,000	$ 250,000		
Subtotal				*$ 927,500*		
Tax	9.5%		$ 927,500	$ 88,113		Sales Tax
Contingency	25%		$ 1,015,613	$ 253,903		
Total Sediment Environmental Controls and Monitoring Cost				$ 1,269,516		

Table D-5 - Alternative 5 Cost Estimate
Quendall Terminals
Renton, Washington

Permeable Treatment Wall						
Mobilization/Demobilization	1	LS	$ 55,765	$ 55,765	Vendor quote	One Pass trencher transport, assembly and disassembly
Excavation and media installation	1	LS	$ 250,000	$ 250,000	Vendor quote	excavate and place GAC
Treatment media	163	ton	$ 920	$ 149,926	Vendor quote	GAC: see Appendix E
Import fill	44	BCY	$ 30	$ 1,333	Project experience	cap for PRB
Monitoring well installation	5	well	$ 4,000	$ 20,000	Project experience	
Transport and Disposal - Non-Hazardous Waste	2,673	ton	$ 50	$ 133,630	project experience	Subtitle D landfill disposal
Transport and Disposal - Hazardous Waste	587	ton	$ 150	$ 88,000	project experience	Subtitle C landfill disposal, assuming no treatment required
Slurry Wall installation	820	LF	$ 188	$ 153,750	Vendor quote	slurry to 25' depth
Subtotal				*$ 852,404*		
Tax	9.5%		$ 852,404	$ 80,978		Sales Tax
Contingency	25%		$ 933,382	$ 233,345		
Total Permeable Treatment Wall Cost				$ 1,166,727		
Subtotal Construction Costs				**$ 36,220,074**		
Professional Services (as percent of construction and contingency costs)						
Project management	5%		$ 36,220,074	$ 1,811,004		
Remedial design	6%		$ 36,220,074	$ 2,173,204		Includes treatability studies for remedy components as necessary
Construction management	6%		$ 36,220,074	$ 2,173,204		
Subtotal				*$ 6,157,413*		
Total Estimated Capital Cost				**$ 42,400,000**		
O&M COSTS						
1st Year O&M						
GW Monitoring	1	LS	$ 80,000	$ 80,000	Project experience	
Sediment Sand Cap and ENR Sampling	1	LS	$ 25,000	$ 25,000	Project experience	
Sediment Cap Inspection	1	LS	$ 15,000	$ 15,000	Project experience	Visual and In-Water (Bathymetric/ Sediment Profile Image)
Backfilled Area Surface Sediment Monitoring	1	LS	$ 25,000	$ 25,000		
DNR Lease	0.5	acre	$ 20,000	$ 10,000		Offshore cap area off property
Subtotal				*$ 155,000*		
Tax	9.5%		$ 155,000	$ 14,725		Sales Tax
Contingency	25%		$ 169,725	$ 42,431		
Total 1st Year O&M Cost				$ 212,156		
Annual O&M						
Groundwater Monitoring	1	LS	$ 25,000	$ 25,000	Project experience	20 wells annually
Upland Cap inspection	6	hour	$ 80	$ 480	labor estimate	
DNR Lease	0.5	acre	$ 20,000	$ 10,000		Offshore cap area off property
Subtotal				*$ 35,480*		
Tax	9.5%		$ 35,480	$ 3,371		Sales Tax
Contingency	25%		$ 38,851	$ 9,713		
Total Annual O&M Cost				$ 48,563		
Professional Services (as percent of Annual O&M costs)						
Project management/Reporting	10%		$ 48,563	$ 4,856		
Total, Annual O&M:				**$ 53,420**		
Total Estimated O&M, 100 Years, No NPV Analysis:				**$ 5,600,000**		
Periodic Costs						
Reactive Cap						
Replace 25% of RC at 22 yrs				$ 110,000		
Replace 25% of RC at 44 yrs				$ 110,000		
Replace 25% of RC at 66 yrs				$ 110,000		
Replace 25% of RC at 88 yrs				$ 110,000		
Sand Cap and ENR						
Sediment Sand Cap and ENR Sampling at 2 years				$ 25,000		
Sediment Sand Cap and ENR Sampling at 5 years				$ 25,000		
Sediment Sand Cap and ENR Sampling at 10 years				$ 25,000		
Sediment Cap Inspection at 2 years				$ 15,000		
Sediment Cap Inspection at 5 years				$ 15,000		
Sediment Cap Inspection at 10 years				$ 15,000		
Sand Cap Shoreline Maintenance at 30 years				$ 25,000		
Sand Cap Shoreline Maintenance at 60 years				$ 25,000		
Sand Cap Shoreline Maintenance at 90 years				$ 25,000		
Permeable treatment wall						
Replace Media at 22 yrs				$ 520,876		includes mob/demob, excavation, media, and $400 per ton disposal fee
Replace Media at 44 yrs				$ 520,876		includes mob/demob, excavation, media, and $400 per ton disposal fee
Replace Media at 66 yrs				$ 520,876		includes mob/demob, excavation, media, and $400 per ton disposal fee
Replace Media at 88 yrs				$ 520,876		includes mob/demob, excavation, media, and $400 per ton disposal fee
Subtotal				*$ 2,718,503*		
TOTAL ESTIMATED COST, NO NPV ANALSYS				**$ 50,718,503**		
Net Present Value Analysis						
Annual O&M	100	year	$ 53,420	$ 2,656,055		
1st year O&M	1	LS	$ 212,156	$ 212,156		
Replace 25% of RC at 22 yrs	1	LS	$ 110,000	$ 77,577		
Replace 25% of RC at 44 yrs	1	LS	$ 110,000	$ 54,710		
Replace 25% of RC at 66 yrs	1	LS	$ 110,000	$ 38,584		
Replace 25% of RC at 88 yrs	1	LS	$ 110,000	$ 27,211		
Sediment Sand Cap and ENR Sampling at 2 years	1	LS	$ 25,000	$ 24,219		
Sediment Sand Cap and ENR Sampling at 5 years	1	LS	$ 25,000	$ 23,093		
Sediment Sand Cap and ENR Sampling at 10 years	1	LS	$ 25,000	$ 21,331		
Sediment Cap Inspection at 2 years	1	LS	$ 15,000	$ 14,531		
Sediment Cap Inspection at 5 years	1	LS	$ 15,000	$ 13,856		
Sediment Cap Inspection at 10 years	1	LS	$ 15,000	$ 12,798		
Sand Cap Shoreline Maintenance at 30 years	1	LS	$ 25,000	$ 15,528		
Sand Cap Shoreline Maintenance at 60 years	1	LS	$ 25,000	$ 9,645		
Sand Cap Shoreline Maintenance at 90 years	1	LS	$ 25,000	$ 5,991		
Replace PRB Media at 22 yrs	1	LS	$ 520,876	$ 367,344		
Replace PRB Media at 44 yrs	1	LS	$ 520,876	$ 259,066		
Replace PRB Media at 66 yrs	1	LS	$ 520,876	$ 182,705		
Replace PRB Media at 88 yrs	1	LS	$ 520,876	$ 128,851		
2013 discount rate for NPV	1.6%					
Total Estimated O&M and Periodic NPV				**$ 4,145,251**		
TOTAL ESTIMATED COST				**$ 46,545,251**		

Notes:
1. Mobilization/Demobilization costs are assumed to include equipment transport and setup, temporary erosion and sedimentation control (TESC) measures, bonds, and insurance.
2. Contingency costs include miscellaneous costs not currently itemized due to the current (preliminary) stage of design development, as well as costs to address unanticipated conditions encountered during construction.

Table D-6 - Alternative 6 Cost Estimate

Quendall Terminals
Renton, Washington

Site:	Quendall Terminals		
Remedial Action Description:	Alternative 6		Containment with Targeted PTM Solidification (RR and MC DNAPL Areas and ≥ 2-Foot-Thickness) and Removal (TD, QP-S, and QP-U DNAPL Areas)
Cost Estimate Accuracy:	FS Screening Level (+50/-30 percent)		
Key Assumptions and Quantities: (see Appendix E for calculations)	Capping of Upland Soil		
	21.6	acre	total area
	940,896	SF	total area
	133,521	SF	permeable area along shoreline
	12,441	BCY	habitat excavation overlap
	104,544	BCY	total volume based on 3' cap thickness
	Enhanced Natural Recovery - Sand Material		
	14,300	BCY	total volume
	Engineered Sand Cap		
	15,800	BCY	total sand volume
	2,150	BCY	removal volume for offsetting sand cap
	40,000	SF	area for offsetting sand cap
	0.5	acre	DNR lease area
	RCM Reactive Capping materials		
	85,600	SF	area of RCM
	1,700	BCY	total sand volume
	570	BCY	removal volume for offsetting reactive cap
	Soil/Sediment Density		
	1.6	tons/BCY	soil density
	1.3	tons/BCY	sediment density
	0.7	tons/CY	organoclay density
	Solidification of Upland Source Area Soil		
	142,501	BCY	volume of soil to be solidified
	133,025	BCY	volume of soil at shallow depths to be solidified
	9,476	BCY	volume of deeper soil to be solidified
	Removal of Upland Source Area Soil		
	12,700	BCY	total volume
	0.5	acre	total area
	2,286	BCY	volume classified as hazardous
	10,414	BCY	volume classified as non-hazardous
	Volume of sediment removal		
	23,200	BCY	sediment removal
	25,900	BCY	total sediment removal volume (including for offsetting cap)
	11,000	BCY	mechanical dredging
	12,200	BCY	hydraulic dredging
	510	BCY	residual cover - organoclay
	2,300	BCY	residual cover - sand
	20,400	BCY	backfill
	35,000	SF	sheet pile area
	Volumes for PRB installation		
	367	BCY	volume classified as hazardous
	1,670	BCY	volume classified as non-hazardous
	163	ton	amount of PRB media
	44	BCY	cover material
	820	LF	slurry wall length
	Dewatering to maintain wet removal for upland soil		
	120	gpm	maximum upland dewatering rate
	120	gpm	average upland dewatering rate
	6	each	deep aquifer depressurization wells
	0.12	year	upland soil removal time
	0.91	year	upland soil solidification time
	16	feet	average excavation depth
	35	feet	min.embed. depth
	10,109	SF	shoring wall area

Item	Quantity	Unit	Unit Cost	Total Cost	Source	Notes
CAPITAL CONSTRUCTION COSTS						
Upland Soil Cap						
Mobilization/Demobilization	1	LS	$ 357,404	$ 357,404	percentage of construction costs	includes temporary facilities for duration of construction
Site Preparation	22	acre	$ 6,900	$ 149,040	Costworks	clearing, grubbing brush and stumps
Geotextile marker layer	104,544	SY	$ 2	$ 158,907	Costworks	non-woven, 120lb tensile strength
Import Fill - Permeable Cap	104,544	BCY	$ 30	$ 3,136,320	project experience	
Compaction	104,544	BCY	$ 5	$ 522,720	project experience	
Habitat Area - excavation	12,441	BCY	$ 6	$ 74,643		
Habitat Area - non-hazardous transport and disposal	19,905	ton	$ 50	$ 995,244		
Hydroseeding	14,836	SY	$ 1	$ 8,901	Costworks	includes seed and fertilizer for wetland area
Stormwater collection and detention system	1,500	LF	$ 40	$ 60,000	project experience	media filter drain
Subtotal				*$ 5,463,180*		
Tax	9.5%		$ 5,463,180	$ 519,002		Sales Tax
Contingency	25%		$ 5,982,183	$ 1,495,546		
Total Upland Soil Cap Cost				$ 7,477,728		
Enhanced Natural Recovery						
Mobilization/Demobilization	1	LS	$ 57,456	$ 57,456		
Sand Material	22,880	ton	$ 20	$ 457,600	vendor quote	
Sand Placement	22,880	ton	$ 15	$ 343,200	project experience	ENR placed as one lift
Confirmation of Placement	1	LS	$ 20,000	$ 20,000		
Subtotal				*$ 878,256*		
Tax	9.5%		$ 878,256	$ 83,434		Sales Tax
Contingency	25%		$ 961,690	$ 240,422.58		
Total Enhanced Natural Recovery Cost				$ 1,202,113		
Engineered Sand Cap						
Mobilization/Demobilization	1	LS	$ 73,584	$ 73,584		
Sand Material	25,280	ton	$ 20	$ 505,600	vendor quote	
Sand Placement	25,280	ton	$ 20	$ 505,600	project experience	Sand Cap placed in multiple lifts
Geotextile Separation Layer	40,000	SF	$ 1	$ 20,000	Vendor quote	Only in nearshore area
Confirmation of Placement	1	LS	$ 20,000	$ 20,000		
Subtotal				*$ 1,124,784*		
Tax	9.5%		$ 1,124,784	$ 106,854		Sales Tax
Contingency	25%		$ 1,231,638	$ 307,910		
Total Engineered Sand Cap Cost				$ 1,539,548		
RCM Reactive Capping						
Mobilization/Demobilization	1	LS	$ 35,627	$ 35,627		
Organoclay RCM Material + Transportation	85,600	SF	$ 3	$ 222,560	Quote from Cetco	
Organoclay RCM Placement	85,600	SF	$ 2	$ 171,200	Project experience	
Sand Material	2,720	ton	$ 20	$ 54,400	vendor quote	
Sand Placement	2,720	ton	$ 15	$ 40,800	project experience	Sand over RCM placed in one lift
Confirmation of Placement	1	LS	$ 20,000	$ 20,000		
Subtotal				*$ 544,587*		
Tax	9.5%		$ 544,587	$ 51,736		Sales Tax
Contingency	25%		$ 596,323	$ 149,081		
Total RCM Reactive Capping Cost				$ 745,404		
Upland Soil Solidification						
Mobilization/Demobilization	1	LS	$ 711,519	$ 711,519	percentage of construction costs	includes temporary facilities for duration of construction
Solidification - 8-ft diameter auger	133,025	BCY	$ 70	$ 9,311,717	project experience	8-ft auger used to cost-effectively treat shallower soils
Solidification - 4-ft diameter auger	9,476	BCY	$ 90	$ 852,847	project experience	4-ft auger used to treat deeper soils, below 8-ft auger limit
Subtotal				*$ 10,876,083*		
Tax	9.5%		$ 10,876,083	$ 1,033,228		Sales Tax
Contingency	30%		$ 11,909,311	$ 3,572,793		
Total Upland Soil Solidification Cost				$ 15,482,104		
Upland Soil Removal						
Mobilization/Demobilization	1	LS	$ 245,881	$ 245,881	percentage of construction costs	includes temporary facilities for duration of construction
Excavation	12,700	BCY	$ 6	$ 76,200	project experience	
Import Fill	12,700	BCY	$ 30	$ 381,000	project experience	
Soil Handling and Stockpiling	12,700	BCY	$ 5	$ 63,500	project experience	segregation into hazardous/non-hazardous
Analytical Sampling	200	ea	$ 500	$ 100,000	project experience	VOCs and SVOCs
Compaction	12,700	BCY	$ 5	$ 63,500	project experience	
Transport and Disposal - Non-Hazardous Waste	16,662	ton	$ 50	$ 833,120	project experience	Subtitle D landfill disposal
Transport and Disposal - Hazardous Waste	3,658	ton	$ 150	$ 548,640	project experience	Subtitle C landfill disposal, assuming no treatment required
Shoring	10,109	SF	$ 92	$ 930,055	project experience	sheet pile - stiffened to allow excavation in the wet (see Appendix F)
Dewatering - Deep Aquifer Depressurization Wells and Pumps	6	ea	$ 40,000	$ 240,000	project experience	
Dewatering - Equalization Tank	2	month	$ 980	$ 1,960	project experience	Rental - 20,000 gallon tank
Dewatering - Treatment system	2	month	$ 8,066	$ 16,132	Vendor quote	rental system: DNAPL separation, air stripping, filtration, GAC vessels
Dewatering - Carbon Replacement	45	day	$ 72	$ 3,198	Vendor quote	based on usage rate of 65 lb/day @ 50gpm - $0.46/lb
Dewatering - Carbon Disposal	3	ton	$ 400	$ 1,391	Vendor quote	
Dewatering - Coagulant	64	lb	$ 2	$ 145	Vendor quote	$2.25 per lb, 1mg/L concentration, average flow rate
Dewatering - Miscellaneous Equipment	20%		$ 363,804	$ 72,761	percentage of dewatering capital co	
Dewatering - Equipment Operation and Maintenance	45	day	$ 700	$ 31,200	labor estimate	1 full-time operator, $70/hr, 10hr/day
Dewatering - Discharge Fee	7,702,062	gal	$ 0	$ 64,697	project experience	$0.0084/gal discharge rate for city of Renton sewer at adjacent site
Dewatering - Power	2	month	$ 2,540	$ 5,080	project experience	$0.0996/KWH estimated power rate
Monitoring Well Installation	20	ea	$ 4,000	$ 80,000	project experience	confirmation monitoring program
Subtotal				*$ 3,758,460*		
Tax	9.5%		$ 3,758,460	$ 357,054		Sales Tax
Contingency	35%		$ 4,115,514	$ 1,440,430		
Total Upland Soil Removal Cost				$ 5,555,943		

Table D-6 - Alternative 6 Cost Estimate

Quendall Terminals
Renton, Washington

Item	Quantity	Unit	Unit Cost	Cost	Source	Notes
Sediment Removal						
Mobilization/Demobilization	1	LS	$ 439,800	$ 439,800		
Mechanical Dredging	13,720	BCY	$ 35	$ 480,194		Mechanical dredging in nearshore and for offsetting nearshore cap
Hydraulic Dredging	12,200	BCY	$ 60	$ 732,000	Project experience	Assumes specialty hydraulic for T-Dock/Offshore
Debris Removal and Disposal	1	LS	$ 50,000	$ 50,000		Removal of piling
Transloading/Material Handling	25,900	BCY	$ 15	$ 388,500		
Dewatering	25,900	BCY	$ 10	$ 246,050	vendor quote	Assumes 5% amendment by weight
Water Treatment	1	LS	$ 200,000	$ 200,000	Project experience	
Residuals Cover Bulk Organoclay Material - (PM-199)	365	ton	$ 3,250	$ 1,185,941	Quote from Cetco	
Residuals Cover Sand Material	3,680	ton	$ 20	$ 73,600	vendor quote	
Residuals Cover Material Placement	4,045	ton	$ 15	$ 60,674	project experience	
Backfill Material	32,640	ton	$ 20	$ 652,800	vendor quote	
Backfill Material Placement	32,640	ton	$ 15	$ 489,600	project experience	Backfill placed in bulk
Transportation and Disposal - Non-Hazardous	33,670	ton	$ 50	$ 1,683,500		Subtitle D landfill disposal
Dredging Confirmation	1	LS	$ 40,000	$ 40,000		
Subtotal				*$ 6,722,659*		
Tax	9.5%		$ 6,722,659	$ 638,653		Sales Tax
Contingency	25%		$ 7,361,312	$ 1,840,328		
Total Sediment Removal Cost				$ 9,201,640		
Sheet Pile Enclosure						
Mobilization/Demobilization	1	LS	$ 220,500	$ 220,500	Project experience	
Steel Unit Cost	35,000	SF	$ 35	$ 1,225,000	Project experience	
Installation Unit Cost	35,000	SF	$ 45	$ 1,575,000	Project experience	
Removal Unit Cost	35,000	SF	$ 15	$ 525,000	Project experience	
Salvage Unit Value	1,750,000	lb	$ (0.1)	$ (175,000)	Project experience	50 pounds per sf
Subtotal				*$ 3,370,500*		
Tax	9.5%		$ 3,370,500	$ 320,198		Sales Tax
Contingency	25%		$ 3,690,698	$ 922,674		
Total Sheet Pile Enclosure Cost				$ 4,613,372		
Sediment Environmental Controls and Monitoring						
Water Quality Monitoring	175	day	$ 2,500	$ 437,500		
Water Quality Controls and BMPs (Absorbent Booms, Silt Curtains, Oil Bo	1	LS	$ 75,000	$ 75,000		
Odor Control	60	day	$ 2,500	$ 150,000		
Noise Monitoring	1	LS	$ 15,000	$ 15,000		
Erosion Protection for Shoreline Area	1	LS	$ 250,000	$ 250,000		
Subtotal				*$ 927,500*		
Tax	9.5%		$ 927,500	$ 88,113		Sales Tax
Contingency	25%		$ 1,015,613	$ 253,903		
Total Sediment Environmental Controls and Monitoring Cost				$ 1,269,516		
Permeable Treatment Wall						
Mobilization/Demobilization	1	LS	$ 55,765	$ 55,765	Vendor quote	One Pass trencher transport, assembly and disassembly
Excavation and media installation	1	LS	$ 250,000	$ 250,000	Vendor quote	excavate and place GAC
Treatment media	163	ton	$ 920	$ 149,926	Vendor quote	GAC: see Appendix E
Import fill	44	BCY	$ 30	$ 1,333	Project experience	cap for PRB
Monitoring well installation	5	well	$ 4,000	$ 20,000	Project experience	
Transport and Disposal - Non-Hazardous Waste	2,673	ton	$ 50	$ 133,630	project experience	Subtitle D landfill disposal
Transport and Disposal - Hazardous Waste	587	ton	$ 150	$ 88,000	project experience	Subtitle C landfill disposal, assuming no treatment required
Slurry Wall installation	820	LF	$ 188	$ 153,750	Vendor quote	slurry to 25' depth
Subtotal				*$ 852,404*		
Tax	9.5%		$ 852,404	$ 80,978		Sales Tax
Contingency	25%		$ 933,382	$ 233,345		
Total Permeable Treatment Wall Cost				$ 1,166,727		
Subtotal Construction Costs				**$ 48,254,095**		
Professional Services (as percent of construction and contingency costs)						
Project management	5%		$ 48,254,095	$ 2,412,705		
Remedial design	6%		$ 48,254,095	$ 2,895,246		Includes treatability studies for remedy components as necessary
Construction management	6%		$ 48,254,095	$ 2,895,246		
Subtotal				*$ 8,203,196*		
Total Estimated Capital Cost				**$ 56,500,000**		
O&M COSTS						
1st Year O&M						
GW Monitoring	1	LS	$ 80,000	$ 80,000	Project experience	
Sediment Sand Cap and ENR Sampling	1	LS	$ 25,000	$ 25,000	Project experience	
Sediment Cap Inspection	1	LS	$ 15,000	$ 15,000	Project experience	Visual and In-Water (Bathymetric/ Sediment Profile Image)
Backfilled Area Surface Sediment Monitoring	1	LS	$ 25,000	$ 25,000		
DNR Lease	0.5	acre	$ 20,000	$ 10,000		Offshore cap area off property
Subtotal				*$ 155,000*		
Tax	9.5%		$ 155,000	$ 14,725		Sales Tax
Contingency	25%		$ 169,725	$ 42,431		
Total 1st Year O&M Cost				$ 212,156		
Annual O&M						
Groundwater Monitoring	1	LS	$ 25,000	$ 25,000	Project experience	20 wells annually
Upland Cap inspection	6	hour	$ 80	$ 480	labor estimate	
DNR Lease	0.5	acre	$ 20,000	$ 10,000		Offshore cap area off property
Subtotal				*$ 35,480*		
Tax	9.5%		$ 35,480	$ 3,371		Sales Tax
Contingency	25%		$ 38,851	$ 9,713		
Total Annual O&M Cost				$ 48,563		
Professional Services (as percent of Annual O&M costs)						
Project management/Reporting	10%		$ 48,563	$ 4,856		
Total, Annual O&M:				**$ 53,420**		
Total Estimated O&M, 100 Years, No NPV Analysis:				**$ 5,600,000**		
Periodic Costs						
Reactive Cap						
Replace 25% of RC at 22 yrs				$ 110,000		
Replace 25% of RC at 44 yrs				$ 110,000		
Replace 25% of RC at 66 yrs				$ 110,000		
Replace 25% of RC at 88 yrs				$ 110,000		
Sand Cap and ENR						
Sediment Sand Cap and ENR Sampling at 2 years				$ 25,000		
Sediment Sand Cap and ENR Sampling at 5 years				$ 25,000		
Sediment Sand Cap and ENR Sampling at 10 years				$ 25,000		
Sediment Cap Inspection at 2 years				$ 15,000		
Sediment Cap Inspection at 5 years				$ 15,000		
Sediment Cap Inspection at 10 years				$ 15,000		
Sand Cap Shoreline Maintenance at 30 years				$ 25,000		
Sand Cap Shoreline Maintenance at 60 years				$ 25,000		
Sand Cap Shoreline Maintenance at 90 years				$ 25,000		
Permeable treatment wall						
Replace Media at 22 yrs				$ 520,876		includes mob/demob, excavation, media, and $400 per ton disposal fee
Replace Media at 44 yrs				$ 520,876		includes mob/demob, excavation, media, and $400 per ton disposal fee
Replace Media at 66 yrs				$ 520,876		includes mob/demob, excavation, media, and $400 per ton disposal fee
Replace Media at 88 yrs				$ 520,876		includes mob/demob, excavation, media, and $400 per ton disposal fee
Subtotal				*$ 2,718,503*		
TOTAL ESTIMATED COST, NO NPV ANALSYS				**$ 64,818,503**		
Net Present Value Analysis						
Annual O&M	100	year	$ 53,420	$ 2,656,055		
1st year O&M	1	LS	$ 212,156	$ 212,156		
Replace 25% of RC at 22 yrs	1	LS	$ 110,000	$ 77,577		
Replace 25% of RC at 44 yrs	1	LS	$ 110,000	$ 54,710		
Replace 25% of RC at 66 yrs	1	LS	$ 110,000	$ 38,584		
Replace 25% of RC at 88 yrs	1	LS	$ 110,000	$ 27,211		
Sediment Sand Cap and ENR Sampling at 2 years	1	LS	$ 25,000	$ 24,219		
Sediment Sand Cap and ENR Sampling at 5 years	1	LS	$ 25,000	$ 23,093		
Sediment Sand Cap and ENR Sampling at 10 years	1	LS	$ 25,000	$ 21,331		
Sediment Cap Inspection at 2 years	1	LS	$ 15,000	$ 14,531		
Sediment Cap Inspection at 5 years	1	LS	$ 15,000	$ 13,856		
Sediment Cap Inspection at 10 years	1	LS	$ 15,000	$ 12,798		
Sand Cap Shoreline Maintenance at 30 years	1	LS	$ 25,000	$ 15,528		
Sand Cap Shoreline Maintenance at 60 years	1	LS	$ 25,000	$ 9,645		
Sand Cap Shoreline Maintenance at 90 years	1	LS	$ 25,000	$ 5,991		
Replace PRB Media at 22 yrs	1	LS	$ 520,876	$ 367,344		
Replace PRB Media at 44 yrs	1	LS	$ 520,876	$ 259,066		
Replace PRB Media at 66 yrs	1	LS	$ 520,876	$ 182,705		
Replace PRB Media at 88 yrs	1	LS	$ 520,876	$ 128,851		
2013 discount rate for NPV	1.6%					
Total Estimated O&M and Periodic NPV				**$ 4,145,251**		
TOTAL ESTIMATED COST				**$ 60,645,251**		

Notes:
1. Mobilization/Demobilization costs are assumed to include equipment transport and setup, temporary erosion and sedimentation control (TESC) measures, bonds, and insurance.
2. Contingency costs include miscellaneous costs not currently itemized due to the current (preliminary) stage of design development, as well as costs to address unanticipated conditions encountered during construction.

Table D-7 - Alternative 7 Cost Estimate

Quendall Terminals
Renton, Washington

Site:	Quendall Terminals
Remedial Action Description:	Alternative 7 Containment with PTM Solidification (Upland) and Removal (Sediment)
Cost Estimate Accuracy:	FS Screening Level (+50/-30 percent)
Key Assumptions and Quantities: (see Appendix E for calculations)	Capping of Upland Soil

Quantity	Unit	Description
Capping of Upland Soil		
21.6	acre	total area
940,896	SF	total area
133,521	SF	permeable area along shoreline
14,836	BCY	habitat excavation overlap
104,544	BCY	total volume based on 3' cap thickness
Enhanced Natural Recovery - Sand Material		
14,300	BCY	total volume
Engineered Sand Cap		
13,600	BCY	total sand volume
1,900	BCY	removal volume for offsetting sand cap
35,000	SF	area for offsetting sand cap
0.3	acre	DNR lease area
Soil/Sediment Density		
1.6	tons/BCY	soil density
1.3	tons/BCY	sediment density
0.7	tons/CY	organoclay density
Solidification of Upland Source Area Soil		
241,275	BCY	volume of soil to be solidified
231,799	BCY	volume of soil at shallow depths to be solidified
9,476	BCY	volume of deeper soil to be solidified
Volume of sediment removal		
56,400	BCY	sediment removal
58,300	BCY	total sediment removal volume (including for offsetting cap)
41,200	BCY	mechanical dredging
15,200	BCY	hydraulic dredging
930	BCY	residual cover - organoclay
4,300	BCY	residual cover - sand
51,200	BCY	backfill
63,000	SF	sheet pile area

Item	Quantity	Unit	Unit Cost	Total Cost	Source	Notes
CAPITAL CONSTRUCTION COSTS						
Upland Soil Cap						
Mobilization/Demobilization	1	LS	$ 371,823	$ 371,823	percentage of construction costs	includes temporary facilities for duration of construction
Site Preparation	22	acre	$ 6,900	$ 149,040	Costworks	clearing, grubbing brush and stumps
Geotextile marker layer	104,544	SY	$ 2	$ 158,907	Costworks	non-woven, 120lb tensile strength
Import Fill - Permeable Cap	104,544	BCY	$ 30	$ 3,136,320	project experience	
Compaction	104,544	BCY	$ 5	$ 522,720	project experience	
Habitat Area - excavation	14,836	BCY	$ 6	$ 89,014		
Habitat Area - non-hazardous transport and disposal	23,737	ton	$ 50	$ 1,186,853		
Hydroseeding	14,836	SY	$ 1	$ 8,901	Costworks	includes seed and fertilizer for wetland area
Stormwater collection and detention system	1,500	LF	$ 40	$ 60,000	project experience	media filter drain
Subtotal				*$ 5,683,579*		
Tax	9.5%		$ 5,683,579	$ 539,940		Sales Tax
Contingency	25%		$ 6,223,518	$ 1,555,880		
Total Upland Soil Cap Cost				$ 7,779,398		
Enhanced Natural Recovery						
Mobilization/Demobilization	1	LS	$ 57,456	$ 57,456		
Sand Material	22,880	ton	$ 20	$ 457,600	vendor quote	
Sand Placement	22,880	ton	$ 15	$ 343,200	project experience	ENR placed as one lift
Confirmation of Placement	1	LS	$ 20,000	$ 20,000		
Subtotal				*$ 878,256*		
Tax	9.5%		$ 878,256	$ 83,434		Sales Tax
Contingency	25%		$ 961,690	$ 240,422.58		
Total Enhanced Natural Recovery Cost				$ 1,202,113		
Engineered Sand Cap						
Mobilization/Demobilization	1	LS	$ 63,553	$ 63,553		
Sand Material	21,760	ton	$ 20	$ 435,200	vendor quote	
Sand Placement	21,760	ton	$ 20	$ 435,200	project experience	Sand Cap placed in multiple lifts
Geotextile Separation Layer	35,000	SF	$ 1	$ 17,500	Vendor quote	Only in nearshore area
Confirmation of Placement	1	LS	$ 20,000	$ 20,000		
Subtotal				*$ 971,453*		
Tax	9.5%		$ 971,453	$ 92,288		Sales Tax
Contingency	25%		$ 1,063,741	$ 265,935		
Total Engineered Sand Cap Cost				$ 1,329,676		
Upland Soil Solidification						
Mobilization/Demobilization	1	LS	$ 1,195,515	$ 1,195,515	percentage of construction costs	includes temporary facilities for duration of construction
Solidification - 8-ft diameter auger	231,799	BCY	$ 70	$ 16,225,938	project experience	8-ft auger used to cost-effectively treat shallower soils
Solidification - 4-ft diameter auger	9,476	BCY	$ 90	$ 852,847	project experience	4-ft auger used to treat deeper soils, below 8-ft auger limit
Subtotal				*$ 18,274,299*		
Tax	9.5%		$ 18,274,299	$ 1,736,058		Sales Tax
Contingency	30%		$ 20,010,358	$ 6,003,107		
Total Upland Soil Solidification Cost				$ 26,013,465		
Sediment Removal						
Mobilization/Demobilization	1	LS	$ 948,775	$ 948,775		
Mechanical Dredging	43,100	BCY	$ 35	$ 1,508,500		Mechanical dredging in nearshore and for offsetting nearshore cap
Hydraulic Dredging	15,200	BCY	$ 60	$ 912,000	Project experience	Assumes specialty hydraulic for T-Dock/Offshore
Debris Removal and Disposal	1	LS	$ 75,000	$ 75,000		Removal of piling
Transloading/Material Handling	58,300	BCY	$ 15	$ 874,500		
Dewatering	58,300	BCY	$ 10	$ 553,850	vendor quote	Assumes 5% amendment by weight
Water Treatment	1	LS	$ 500,000	$ 500,000	Project experience	
Residuals Cover Bulk Organoclay Material - (PM-199)	665	ton	$ 3,250	$ 2,162,599	Quote from Cetco	
Residuals Cover Sand Material	6,880	ton	$ 20	$ 137,600	vendor quote	
Residuals Cover Material Placement	7,545	ton	$ 15	$ 113,181	project experience	
Backfill Material	81,920	ton	$ 20	$ 1,638,400	vendor quote	
Backfill Material Placement	81,920	ton	$ 15	$ 1,228,800	project experience	Backfill placed in bulk
Transportation and Disposal - Non-Hazardous	75,790	ton	$ 50	$ 3,789,500		Subtitle D landfill disposal
Dredging Confirmation	1	LS	$ 60,000	$ 60,000		
Subtotal				*$ 14,502,705*		
Tax	9.5%		$ 14,502,705	$ 1,377,757		Sales Tax
Contingency	25%		$ 15,880,462	$ 3,970,116		
Total Sediment Removal Cost				$ 19,850,578		

Table D-7 - Alternative 7 Cost Estimate
Quendall Terminals
Renton, Washington

Sheet Pile Enclosure						
Mobilization/Demobilization	1	LS	$ 396,900	$ 396,900	Project experience	
Steel Unit Cost	63,000	SF	$ 35	$ 2,205,000	Project experience	
Installation Unit Cost	63,000	SF	$ 45	$ 2,835,000	Project experience	
Removal Unit Cost	63,000	SF	$ 15	$ 945,000	Project experience	
Salvage Unit Value	3,150,000	lb	$ (0.1)	$ (315,000)	Project experience	50 pounds per sf
Subtotal				*$ 6,066,900*		
Tax	9.5%		$ 6,066,900	$ 576,356		Sales Tax
Contingency	25%		$ 6,643,256	$ 1,660,814		
Total Sheet Pile Enclosure Cost				$ 8,304,069		
Sediment Environmental Controls and Monitoring						
Water Quality Monitoring	250	day	$ 2,500	$ 625,000		
Water Quality Controls and BMPs (Absorbent Booms, Silt Curtains, Oil Bo	1	LS	$ 150,000	$ 150,000		
Odor Control	150	day	$ 2,500	$ 375,000		
Noise Monitoring	1	LS	$ 30,000	$ 30,000		
Erosion Protection for Shoreline Area	1	LS	$ 250,000	$ 250,000		
Subtotal				*$ 1,430,000*		
Tax	9.5%		$ 1,430,000	$ 135,850		Sales Tax
Contingency	25%		$ 1,565,850	$ 391,463		
Total Sediment Environmental Controls and Monitoring Cost				$ 1,957,313		
Subtotal Construction Costs				**$ 66,436,612**		
Professional Services (as percent of construction and contingency costs)						
Project management	5%		$ 66,436,612	$ 3,321,831		
Remedial design	6%		$ 66,436,612	$ 3,986,197		Includes treatability studies for remedy components as necessary
Construction management	6%		$ 66,436,612	$ 3,986,197		
Subtotal				*$ 11,294,224*		
Total Estimated Capital Cost				**$ 77,700,000**		
O&M COSTS						
1st Year O&M						
GW Monitoring	1	LS	$ 80,000	$ 80,000	Project experience	
Sediment Sand Cap and ENR Sampling	1	LS	$ 25,000	$ 25,000	Project experience	
Sediment Cap Inspection	1	LS	$ 15,000	$ 15,000	Project experience	Visual and In-Water (Bathymetric/ Sediment Profile Image)
Backfilled Area Surface Sediment Monitoring	1	LS	$ 25,000	$ 25,000		
DNR Lease	0.3	acre	$ 20,000	$ 6,000		Offshore cap area off property
Subtotal				*$ 151,000*		
Tax	9.5%		$ 151,000	$ 14,345		Sales Tax
Contingency	25%		$ 165,345	$ 41,336		
Total 1st Year O&M Cost				$ 206,681		
Annual O&M						
Groundwater Monitoring	1	LS	$ 25,000	$ 25,000	Project experience	20 wells annually
Upland Cap inspection	6	hour	$ 80	$ 480	labor estimate	
DNR Lease	0.3	acre	$ 20,000	$ 6,000		Offshore cap area off property
Subtotal				*$ 31,480*		
Tax	9.5%		$ 31,480	$ 2,991		Sales Tax
Contingency	25%		$ 34,471	$ 8,618		
Total Annual O&M Cost				$ 43,088		
Professional Services (as percent of Annual O&M costs)						
Project management/Reporting	10%		$ 43,088	$ 4,309		
Total, Annual O&M:				**$ 47,397**		
Total Estimated O&M, 100 Years, No NPV Analysis:				**$ 4,900,000**		
Periodic Costs						
Sand Cap and ENR						
Sediment Sand Cap and ENR Sampling at 2 years				$ 25,000		
Sediment Sand Cap and ENR Sampling at 5 years				$ 25,000		
Sediment Sand Cap and ENR Sampling at 10 years				$ 25,000		
Sediment Cap Inspection at 2 years				$ 15,000		
Sediment Cap Inspection at 5 years				$ 15,000		
Sediment Cap Inspection at 10 years				$ 15,000		
Sand Cap Shoreline Maintenance at 30 years				$ 25,000		
Sand Cap Shoreline Maintenance at 60 years				$ 25,000		
Sand Cap Shoreline Maintenance at 90 years				$ 25,000		
Subtotal				*$ 195,000*		
TOTAL ESTIMATED COST, NO NPV ANALSYS				**$ 82,795,000**		
Net Present Value Analysis						
Annual O&M	100	year	$ 47,397	$ 2,356,613		
1st year O&M	1	LS	$ 206,681	$ 206,681		
Sediment Sand Cap and ENR Sampling at 2 years	1	LS	$ 25,000	$ 24,219		
Sediment Sand Cap and ENR Sampling at 5 years	1	LS	$ 25,000	$ 23,093		
Sediment Sand Cap and ENR Sampling at 10 years	1	LS	$ 25,000	$ 21,331		
Sediment Cap Inspection at 2 years	1	LS	$ 15,000	$ 14,531		
Sediment Cap Inspection at 5 years	1	LS	$ 15,000	$ 13,856		
Sediment Cap Inspection at 10 years	1	LS	$ 15,000	$ 12,798		
Sand Cap Shoreline Maintenance at 30 years	1	LS	$ 25,000	$ 15,528		
Sand Cap Shoreline Maintenance at 60 years	1	LS	$ 25,000	$ 9,645		
Sand Cap Shoreline Maintenance at 90 years	1	LS	$ 25,000	$ 5,991		
2013 discount rate for NPV	1.6%					
Total Estimated O&M and Periodic NPV				**$ 2,704,286**		
TOTAL ESTIMATED COST				**$ 80,404,286**		

Notes:
1. Mobilization/Demobilization costs are assumed to include equipment transport and setup, temporary erosion and sedimentation control (TESC) measures, bonds, and insurance.
2. Contingency costs include miscellaneous costs not currently itemized due to the current (preliminary) stage of design development, as well as costs to address unanticipated conditions encountered during construction.

Table D-8 - Alternative 8 Cost Estimate

Quendall Terminals
Renton, Washington

Site:	Quendall Terminals		
Remedial Action Description:	Alternative 8		Containment with PTM Removal (Upland and Sediment)
Cost Estimate Accuracy:	FS Screening Level (+50/-30 percent)		
Key Assumptions and Quantities:	Capping of Upland Soil		
(see Appendix E for calculations)	21.6	acre	total area
	940,896	SF	total area
	133,521	SF	permeable area along shoreline
	9,721	BCY	habitat excavation overlap
	104,544	BCY	total volume based on 3' cap thickness
	Enhanced Natural Recovery - Sand Material		
	14,300	BCY	total volume
	Engineered Sand Cap		
	13,600	BCY	total sand volume
	1,900	BCY	removal volume for offsetting sand cap
	35,000	SF	area for offsetting sand cap
	0.3	acre	DNR lease area
	Soil/Sediment Density		
	1.6	tons/BCY	soil density
	1.3	tons/BCY	sediment density
	0.7	tons/CY	organoclay density
	Removal of Upland Source Area Soil		
	210,100	BCY	total volume
	9.7	acre	total area
	30,474	BCY	volume classified as hazardous
	179,626	BCY	volume classified as non-hazardous
	Volume of sediment removal		
	56,400	BCY	sediment removal
	58,300	BCY	total sediment removal volume (including for offsetting cap)
	41,200	BCY	mechanical dredging
	15,200	BCY	hydraulic dredging
	930	BCY	residual cover - organoclay
	4,300	BCY	residual cover - sand
	51,200	BCY	backfill
	63,000	SF	sheet pile area
	Dewatering to maintain wet removal for upland soil		
	207	gpm	maximum upland dewatering rate
	67	gpm	average upland dewatering rate
	27	each	deep aquifer depressurization wells
	2.02	year	upland soil removal time
	19	feet	average excavation depth
	30	feet	min.embed. depth
	127,809	SF	shoring wall area

Item	Quantity	Unit	Unit Cost	Total Cost	Source	Notes
CAPITAL CONSTRUCTION COSTS						
Upland Soil Cap						
Mobilization/Demobilization	1	LS	$ 292,312	$ 292,312	percentage of construction costs	includes temporary facilities for duration of construction
Site Preparation	22	acre	$ 6,900	$ 149,040	Costworks	clearing, grubbing brush and stumps
Geotextile marker layer	104,544	SY	$ 2	$ 158,907	Costworks	non-woven, 120lb tensile strength
Import Fill - Permeable Cap	104,544	BCY	$ 30	$ 3,136,320	project experience	
Compaction	104,544	BCY	$ 5	$ 522,720	project experience	
Habitat Area - excavation	9,721	BCY	$ 6	$ 58,324		
Habitat Area - non-hazardous transport and disposal	15,553	ton	$ 50	$ 777,653		
Hydroseeding	14,836	SY	$ 1	$ 8,901	Costworks	includes seed and fertilizer for wetland area
Stormwater collection and detention system	1,500	LF	$ 40	$ 60,000	project experience	media filter drain
Subtotal				*$ 5,164,178*		
Tax	9.5%		$ 5,164,178	$ 490,597		Sales Tax
Contingency	25%		$ 5,654,774	$ 1,413,694		
Total Upland Soil Cap Cost				$ 7,068,468		
Enhanced Natural Recovery						
Mobilization/Demobilization	1	LS	$ 49,248	$ 49,248		
Sand Material	22,880	ton	$ 20	$ 457,600	vendor quote	
Sand Placement	22,880	ton	$ 15	$ 343,200	project experience	ENR placed as one lift
Confirmation of Placement	1	LS	$ 20,000	$ 20,000		
Subtotal				*$ 870,048*		
Tax	9.5%		$ 870,048	$ 82,655		Sales Tax
Contingency	25%		$ 952,703	$ 238,175.64		
Total Enhanced Natural Recovery Cost				$ 1,190,878		
Engineered Sand Cap						
Mobilization/Demobilization	1	LS	$ 54,474	$ 54,474		
Sand Material	21,760	ton	$ 20	$ 435,200	vendor quote	
Sand Placement	21,760	ton	$ 20	$ 435,200	project experience	Sand Cap placed in multiple lifts
Geotextile Separation Layer	35,000	SF	$ 1	$ 17,500	Vendor quote	Only in nearshore area
Confirmation of Placement	1	LS	$ 20,000	$ 20,000		
Subtotal				*$ 962,374*		
Tax	9.5%		$ 962,374	$ 91,426		Sales Tax
Contingency	25%		$ 1,053,800	$ 263,450		
Total Engineered Sand Cap Cost				$ 1,317,249		
Upland Soil Removal						
Mobilization/Demobilization	1	LS	$ 2,974,731	$ 2,974,731	percentage of construction costs	includes temporary facilities for duration of construction
Excavation	210,100	BCY	$ 6	$ 1,260,600	project experience	
Soil Handling and Stockpiling	210,100	BCY	$ 5	$ 1,050,500	project experience	segregation into hazardous/non-hazardous
Analytical Sampling	200	ea	$ 500	$ 100,000	project experience	VOCs and SVOCs
Compaction	210,100	BCY	$ 5	$ 1,050,500	project experience	
On-Site Treatment - Thermal Desorption	336,160	ton	$ 95	$ 31,935,200	vendor estimate	
Shoring	127,809	SF	$ 92	$ 11,758,426	project experience	sheet pile - stiffened to allow excavation in the wet (see Appendix F)
Dewatering - Deep Aquifer Depressurization Wells and Pumps	27	ea	$ 40,000	$ 1,080,000	project experience	
Dewatering - Equalization Tank	25	month	$ 980	$ 24,500	project experience	Rental - 20,000 gallon tank
Dewatering - Treatment system	25	month	$ 8,066	$ 201,650	Vendor quote	rental system: DNAPL separation, air stripping, filtration, GAC vessels
Dewatering - Arsenic Treatment and Media	1	LS	$ 23,071	$ 23,071	Vendor quote	based on usage rate of 4% by weight
Dewatering - Carbon Replacement	737	day	$ 40	$ 29,499	Vendor quote	based on usage rate of 65 lb/day @ 50gpm - $0.46/lb
Dewatering - Carbon Disposal	32	ton	$ 400	$ 12,826	Vendor quote	
Dewatering - Coagulant	593	lb	$ 2	$ 1,335	Vendor quote	$2.25 per lb, 1mg/L concentration, average flow rate
Dewatering - Miscellaneous Equipment	20%		$ 1,953,025	$ 390,605	percentage of dewatering capital co	
Dewatering - Equipment Operation and Maintenance	737	day	$ 700	$ 516,159	labor estimate	1 full-time operator, $70/hr, 10hr/day
Dewatering - Power	25	month	$ 2,540	$ 63,500	project experience	$0.0996/KWH estimated power rate
Dewatering - Outfall Piping	50	LF	$ 10	$ 486	Costworks	8" Concrete discharge pipe
Monitoring Well Installation	20	ea	$ 4,000	$ 80,000	project experience	confirmation monitoring program
Subtotal				*$ 52,553,588*		
Tax	9.5%		$ 52,553,588	$ 4,992,591		Sales Tax
Contingency	35%		$ 57,546,179	$ 20,141,163		
Total Upland Soil Removal Cost				$ 77,687,341		
Sediment Removal						
Mobilization/Demobilization	1	LS	$ 813,236	$ 813,236		
Mechanical Dredging	43,100	BCY	$ 35	$ 1,508,500		Mechanical dredging in nearshore and for offsetting nearshore cap
Hydraulic Dredging	15,200	BCY	$ 60	$ 912,000	Project experience	Assumes specialty hydraulic for T-Dock/Offshore
Debris Removal and Disposal	1	LS	$ 75,000	$ 75,000		Removal of piling
Transloading/Material Handling	58,300	BCY	$ 15	$ 874,500		
Dewatering	58,300	BCY	$ 10	$ 553,850	vendor quote	Assumes 5% amendment by weight
Water Treatment	1	LS	$ 500,000	$ 500,000	Project experience	
Residuals Cover Bulk Organoclay Material - (PM-199)	665	ton	$ 3,250	$ 2,162,599	Quote from Cetco	
Residuals Cover Sand Material	6,880	ton	$ 20	$ 137,600	vendor quote	
Residuals Cover Material Placement	7,545	ton	$ 15	$ 113,181	project experience	
Backfill Material	81,920	ton	$ 20	$ 1,638,400	vendor quote	
Backfill Material Placement	81,920	ton	$ 15	$ 1,228,800	project experience	Backfill placed in bulk
Transportation and Disposal - Non-Hazardous	75,790	ton	$ 50	$ 3,789,500		Subtitle D landfill disposal
Dredging Confirmation	1	LS	$ 60,000	$ 60,000		
Subtotal				*$ 14,367,166*		
Tax	9.5%		$ 14,367,166	$ 1,364,881		Sales Tax
Contingency	25%		$ 15,732,047	$ 3,933,012		
Total Sediment Removal Cost				$ 19,665,058		
Sheet Pile Enclosure						
Mobilization/Demobilization	1	LS	$ 340,200	$ 340,200	Project experience	
Steel Unit Cost	63,000	SF	$ 35	$ 2,205,000	Project experience	
Installation Unit Cost	63,000	SF	$ 45	$ 2,835,000	Project experience	
Removal Unit Cost	63,000	SF	$ 15	$ 945,000	Project experience	
Salvage Unit Value	3,150,000	lb	$ (0.1)	$ (315,000)	Project experience	50 pounds per sf
Subtotal				*$ 6,010,200*		
Tax	9.5%		$ 6,010,200	$ 570,969		Sales Tax
Contingency	25%		$ 6,581,169	$ 1,645,292		
Total Sheet Pile Enclosure Cost				$ 8,226,461		

Table D-8 - Alternative 8 Cost Estimate
Quendall Terminals
Renton, Washington

Sediment Environmental Controls and Monitoring						
Water Quality Monitoring	250	day	$ 2,500	$ 625,000		
Water Quality Controls and BMPs (Absorbent Booms, Silt Curtains, Oil Bo	1	LS	$ 150,000	$ 150,000		
Odor Control	150	day	$ 2,500	$ 375,000		
Noise Monitoring	1	LS	$ 30,000	$ 30,000		
Erosion Protection for Shoreline Area	1	LS	$ 250,000	$ 250,000		
Subtotal				*$ 1,430,000*		
Tax	9.5%		$ 1,430,000	$ 135,850		Sales Tax
Contingency	25%		$ 1,565,850	$ 391,463		
Total Sediment Environmental Controls and Monitoring Cost				$ 1,957,313		
Subtotal Construction Costs				**$ 117,112,769**		
Professional Services (as percent of construction and contingency costs)						
Project management	5%		$ 117,112,769	$ 5,855,638		
Remedial design	6%		$ 117,112,769	$ 7,026,766		Includes treatability studies for remedy components as necessary
Construction management	6%		$ 117,112,769	$ 7,026,766		
Subtotal				*$ 19,909,171*		
Total Estimated Capital Cost				**$ 137,000,000**		
O&M COSTS						
1st Year O&M						
GW Monitoring	1	LS	$ 80,000	$ 80,000	Project experience	
Sediment Sand Cap and ENR Sampling	1	LS	$ 25,000	$ 25,000	Project experience	
Sediment Cap Inspection	1	LS	$ 15,000	$ 15,000	Project experience	Visual and In-Water (Bathymetric/ Sediment Profile Image)
Backfilled Area Surface Sediment Monitoring	1	LS	$ 25,000	$ 25,000		
DNR Lease	0.3	acre	$ 20,000	$ 6,000		Offshore cap area off property
Subtotal				*$ 151,000*		
Tax	9.5%		$ 151,000	$ 14,345		Sales Tax
Contingency	25%		$ 165,345	$ 41,336		
Total 1st Year O&M Cost				$ 206,681		
Annual O&M						
Groundwater Monitoring	1	LS	$ 25,000	$ 25,000	Project experience	20 wells annually
Upland Cap inspection	6	hour	$ 80	$ 480	labor estimate	
DNR Lease	0	acre	$ 20,000	$ 6,000		Offshore cap area off property
Subtotal				*$ 31,480*		
Tax	9.5%		$ 31,480	$ 2,991		Sales Tax
Contingency	25%		$ 34,471	$ 8,618		
Total Annual O&M Cost				$ 43,088		
Professional Services (as percent of Annual O&M costs)						
Project management/Reporting	10%		$ 43,088	$ 4,309		
Total, Annual O&M:				**$ 47,397**		
Total Estimated O&M, 100 Years, No NPV Analysis:				**$ 4,900,000**		
Periodic Costs						
Sand Cap and ENR						
Sediment Sand Cap and ENR Sampling at 2 years				$ 25,000		
Sediment Sand Cap and ENR Sampling at 5 years				$ 25,000		
Sediment Sand Cap and ENR Sampling at 10 years				$ 25,000		
Sediment Cap Inspection at 2 years				$ 15,000		
Sediment Cap Inspection at 5 years				$ 15,000		
Sediment Cap Inspection at 10 years				$ 15,000		
Sand Cap Shoreline Maintenance at 30 years				$ 25,000		
Sand Cap Shoreline Maintenance at 60 years				$ 25,000		
Sand Cap Shoreline Maintenance at 90 years				$ 25,000		
Subtotal				*$ 195,000*		
TOTAL ESTIMATED COST, NO NPV ANALSYS				**$ 142,095,000**		
Net Present Value Analysis						
Annual O&M	100	year	$ 47,397	$ 2,356,613		
1st year O&M	1	LS	$ 206,681	$ 206,681		
Sediment Sand Cap and ENR Sampling at 2 years	1	LS	$ 25,000	$ 24,219		
Sediment Sand Cap and ENR Sampling at 5 years	1	LS	$ 25,000	$ 23,093		
Sediment Sand Cap and ENR Sampling at 10 years	1	LS	$ 25,000	$ 21,331		
Sediment Cap Inspection at 2 years	1	LS	$ 15,000	$ 14,531		
Sediment Cap Inspection at 5 years	1	LS	$ 15,000	$ 13,856		
Sediment Cap Inspection at 10 years	1	LS	$ 15,000	$ 12,798		
Sand Cap Shoreline Maintenance at 30 years	1	LS	$ 25,000	$ 15,528		
Sand Cap Shoreline Maintenance at 60 years	1	LS	$ 25,000	$ 9,645		
Sand Cap Shoreline Maintenance at 90 years	1	LS	$ 25,000	$ 5,991		
2013 discount rate for NPV	1.6%					
Total Estimated O&M and Periodic NPV				**$ 2,704,286**		
TOTAL ESTIMATED COST				**$ 139,704,286**		

Notes:
1. Mobilization/Demobilization costs are assumed to include equipment transport and setup, temporary erosion and sedimentation control (TESC) measures, bonds, and insurance.
2. Contingency costs include miscellaneous costs not currently itemized due to the current (preliminary) stage of design development, as well as costs to address unanticipated conditions encountered during construction.

Table D-9 - Alternative 9 Cost Estimate

Quendall Terminals
Renton, Washington

Site:	Quendall Terminals		
Remedial Action Description:	Alternative	9	Containment with Solidfication and Removal of Contaminated Soil and Removal of Contaminated Sediment
Cost Estimate Accuracy:	FS Screening Level (+50/-30 percent)		
Key Assumptions and Quantities:	Capping of Upland Soil		
(see Appendix E for calculations)	21.6	acre	total area
	940,896	SF	total area
	133,521	SF	permeable area along shoreline
	5,023	BCY	habitat excavation overlap
	104,544	BCY	total volume based on 3' cap thickness
	Enhanced Natural Recovery - Sand Material		
	14,300	BCY	total volume
	Engineered Sand Cap		
	9,700	BCY	total sand volume
	800	BCY	removal volume for offsetting sand cap
	15,000	SF	area for offsetting sand cap
	0.3	acre	DNR lease area
	Soil/Sediment Density		
	1.6	tons/BCY	soil density
	1.3	tons/BCY	sediment density
	0.7	tons/CY	organoclay density
	Solidification of Upland Source Area Soil		
	362,900	BCY	volume of soil to be solidified
	285,901	BCY	volume of soil at shallow depths to be solidified
	76,999	BCY	volume of deeper soil to be solidified
	Removal of Upland Source Area Soil		
	342,500	BCY	total volume
	14.1	acre	total area
	Volume of sediment removal		
	172,300	BCY	sediment removal
	173,100	BCY	total sediment removal volume (including for offsetting cap)
	148,600	BCY	mechanical dredging
	23,700	BCY	hydraulic dredging
	1,170	BCY	residual cover - organoclay
	5,400	BCY	residual cover - sand
	165,900	BCY	backfill
	91,860	SF	sheet pile area
	Dewatering to maintain wet removal for upland soil		
	289	gpm	maximum upland dewatering rate
	70	gpm	average upland dewatering rate
	60	each	deep aquifer depressurization wells
	3.29	year	upland soil removal time
	2.33	year	upland soil solidification time
	15	feet	average excavation depth
	35	feet	min.embed. depth
	101,385	SF	shoring wall area

Item	Quantity	Unit	Unit Cost	Total Cost	Source	Notes
CAPITAL CONSTRUCTION COSTS						
Upland Soil Cap						
Mobilization/Demobilization	1	LS	$ 268,073	$ 268,073	percentage of construction costs	includes temporary facilities for duration of construction
Site Preparation	22	acre	$ 6,900	$ 149,040	Costworks	clearing, grubbing brush and stumps
Geotextile marker layer	104,544	SY	$ 2	$ 158,907	Costworks	non-woven, 120lb tensile strength
Import Fill - Permeable Cap	104,544	BCY	$ 30	$ 3,136,320	project experience	
Compaction	104,544	BCY	$ 5	$ 522,720	project experience	
Habitat Area - excavation	5,023	BCY	$ 6	$ 30,139		
Habitat Area - non-hazardous transport and disposal	8,037	ton	$ 50	$ 401,858		
Hydroseeding	14,836	SY	$ 1	$ 8,901	Costworks	includes seed and fertilizer for wetland area
Stormwater collection and detention system	1,500	LF	$ 40	$ 60,000	project experience	media filter drain
Subtotal				*$ 4,735,959*		
Tax	9.5%		$ 4,735,959	$ 449,916		Sales Tax
Contingency	25%		$ 5,185,875	$ 1,296,469		
Total Upland Soil Cap Cost				$ 6,482,343		
Enhanced Natural Recovery						
Mobilization/Demobilization	1	LS	$ 49,248	$ 49,248		
Sand Material	22,880	ton	$ 20	$ 457,600	vendor quote	
Sand Placement	22,880	ton	$ 15	$ 343,200	project experience	ENR placed as one lift
Confirmation of Placement	1	LS	$ 20,000	$ 20,000		
Subtotal				*$ 870,048*		
Tax	9.5%		$ 870,048	$ 82,655		Sales Tax
Contingency	25%		$ 952,703	$ 238,175.64		
Total Enhanced Natural Recovery Cost				$ 1,190,878		
Engineered Sand Cap						
Mobilization/Demobilization	1	LS	$ 38,898	$ 38,898		
Sand Material	15,520	ton	$ 20	$ 310,400	vendor quote	
Sand Placement	15,520	ton	$ 20	$ 310,400	project experience	Sand Cap placed in multiple lifts
Geotextile Separation Layer	15,000	SF	$ 1	$ 7,500	Vendor quote	Only in nearshore area
Confirmation of Placement	1	LS	$ 20,000	$ 20,000		
Subtotal				*$ 687,198*		
Tax	9.5%		$ 687,198	$ 65,284		Sales Tax
Contingency	25%		$ 752,482	$ 188,120		
Total Engineered Sand Cap Cost				$ 940,602		
Upland Soil Solidification						
Mobilization/Demobilization	1	LS	$ 1,616,579	$ 1,616,579	percentage of construction costs	includes temporary facilities for duration of construction
Solidification - 8-ft diameter auger	285,901	BCY	$ 70	$ 20,013,065	project experience	8-ft auger used to cost-effectively treat shallower soils
Solidification - 4-ft diameter auger	76,999	BCY	$ 90	$ 6,929,917	project experience	4-ft auger used to treat deeper soils, below 8-ft auger limit
Subtotal				*$ 28,559,560*		
Tax	9.5%		$ 28,559,560	$ 2,713,158		Sales Tax
Contingency	30%		$ 31,272,719	$ 9,381,816		
Total Upland Soil Solidification Cost				$ 40,654,534		
Upland Soil Removal						
Mobilization/Demobilization	1	LS	$ 4,108,079	$ 4,108,079	percentage of construction costs	includes temporary facilities for duration of construction
Excavation	342,500	BCY	$ 6	$ 2,055,000	project experience	
Soil Handling and Stockpiling	342,500	BCY	$ 5	$ 1,712,500	project experience	segregation into hazardous/non-hazardous
Analytical Sampling	200	ea	$ 500	$ 100,000	project experience	VOCs and SVOCs
Compaction	342,500	BCY	$ 5	$ 1,712,500	project experience	
On-Site Treatment - Thermal Desorption	548,000	ton	$ 95	$ 52,060,000	vendor estimate	
Shoring	101,385	SF	$ 61	$ 6,184,485	project experience	sheet pile (see Appendix F)
Dewatering - Deep Aquifer Depressurization Wells and Pumps	60	ea	$ 40,000	$ 2,400,000	project experience	
Dewatering - Equalization Tank	40	month	$ 980	$ 39,200	project experience	Rental - 20,000 gallon tank
Dewatering - Treatment system	40	month	$ 8,066	$ 322,640	Vendor quote	rental system: DNAPL separation, air stripping, filtration, GAC vessels
Dewatering - Arsenic Treatment and Media	1	LS	$ 23,071	$ 23,071	Vendor quote	based on usage rate of 4% by weight
Dewatering - Carbon Replacement	1,202	day	$ 42	$ 50,328	Vendor quote	based on usage rate of 65 lb/day @ 50gpm - $0.46/lb
Dewatering - Carbon Disposal	55	ton	$ 400	$ 21,882	Vendor quote	
Dewatering - Coagulant	1,012	lb	$ 2	$ 2,277	Vendor quote	$2.25 per lb, 1mg/L concentration, average flow rate
Dewatering - Miscellaneous Equipment	20%		$ 3,802,913	$ 760,583	percentage of dewatering capital co	
Dewatering - Equipment Operation and Maintenance	1,202	day	$ 700	$ 841,430	labor estimate	1 full-time operator, $70/hr, 10hr/day
Dewatering - Power	40	month	$ 2,540	$ 101,600	project experience	$0.0996/KWH estimated power rate
Dewatering - Outfall Piping	50	LF	$ 10	$ 486	Costworks	8" Concrete discharge pipe
Monitoring Well Installation	20	ea	$ 4,000	$ 80,000	project experience	confirmation monitoring program
Subtotal				*$ 72,576,060*		
Tax	9.5%		$ 72,576,060	$ 6,894,726		Sales Tax
Contingency	35%		$ 79,470,785	$ 27,814,775		
Total Upland Soil Removal Cost				$ 107,285,560		
Sediment Removal						
Mobilization/Demobilization	1	LS	$ 2,106,270	$ 2,106,270		
Mechanical Dredging	149,400	BCY	$ 35	$ 5,229,000		Mechanical dredging in nearshore and for offsetting nearshore cap
Hydraulic Dredging	23,700	BCY	$ 60	$ 1,422,000	Project experience	Assumes specialty hydraulic for T-Dock/Offshore
Debris Removal and Disposal	1	LS	$ 75,000	$ 75,000		Removal of piling
Transloading/Material Handling	173,100	BCY	$ 15	$ 2,596,500		
Dewatering	173,100	BCY	$ 10	$ 1,644,450	vendor quote	Assumes 5% amendment by weight
Water Treatment	1	LS	$ 500,000	$ 500,000	Project experience	
Residuals Cover Bulk Organoclay Material - (PM-199)	837	ton	$ 3,250	$ 2,720,689	Quote from Cetco	
Residuals Cover Sand Material	8,640	ton	$ 20	$ 172,800	vendor quote	
Residuals Cover Material Placement	9,477	ton	$ 15	$ 142,157	project experience	
Backfill Material	265,440	ton	$ 20	$ 5,308,800	vendor quote	
Backfill Material Placement	265,440	ton	$ 15	$ 3,981,600	project experience	Backfill placed in bulk
Transportation and Disposal - Non-Hazardous	225,030	ton	$ 50	$ 11,251,500		Subtitle D landfill disposal
Dredging Confirmation	1	LS	$ 60,000	$ 60,000		
Subtotal				*$ 37,210,766*		
Tax	9.5%		$ 37,210,766	$ 3,535,023		Sales Tax
Contingency	25%		$ 40,745,788	$ 10,186,447		
Total Sediment Removal Cost				$ 50,932,235		

Table D-9 - Alternative 9 Cost Estimate
Quendall Terminals
Renton, Washington

Item	Quantity	Unit	Unit Cost	Cost	Source	Notes
Sheet Pile Enclosure						
Mobilization/Demobilization	1	LS	$ 496,044	$ 496,044	Project experience	
Steel Unit Cost	91,860	SF	$ 35	$ 3,215,100	Project experience	
Installation Unit Cost	91,860	SF	$ 45	$ 4,133,700	Project experience	
Removal Unit Cost	91,860	SF	$ 15	$ 1,377,900	Project experience	
Salvage Unit Value	4,593,000	lb	$ (0.1)	$ (459,300)	Project experience	50 pounds per sf
Subtotal				*$ 8,763,444*		
Tax	9.5%		$ 8,763,444	$ 832,527		Sales Tax
Contingency	25%		$ 9,595,971	$ 2,398,993		
Total Sheet Pile Enclosure Cost				$ 11,994,964		
Sediment Environmental Controls and Monitoring						
Water Quality Monitoring	250	day	$ 2,500	$ 625,000		
Water Quality Controls and BMPs (Absorbent Booms, Silt Curtains, Oil Bo	1	LS	$ 200,000	$ 200,000		
Odor Control	220	day	$ 2,500	$ 550,000		
Noise Monitoring	1	LS	$ 30,000	$ 30,000		
Erosion Protection for Shoreline Area	1	LS	$ 250,000	$ 250,000		
Subtotal				*$ 1,655,000*		
Tax	9.5%		$ 1,655,000	$ 157,225		Sales Tax
Contingency	25%		$ 1,812,225	$ 453,056		
Total Sediment Environmental Controls and Monitoring Cost				$ 2,265,281		
Subtotal Construction Costs				**$ 221,746,399**		
Professional Services (as percent of construction and contingency costs)						
Project management	5%		$ 221,746,399	$ 11,087,320		
Remedial design	6%		$ 221,746,399	$ 13,304,784		Includes treatability studies for remedy components as necessary
Construction management	6%		$ 221,746,399	$ 13,304,784		
Subtotal				*$ 37,696,888*		
Total Estimated Capital Cost				**$ 259,400,000**		
O&M COSTS						
1st Year O&M						
GW Monitoring	1	LS	$ 80,000	$ 80,000	Project experience	
Sediment Sand Cap and ENR Sampling	1	LS	$ 25,000	$ 25,000	Project experience	
Sediment Cap Inspection	1	LS	$ 15,000	$ 15,000	Project experience	Visual and In-Water (Bathymetric/ Sediment Profile Image)
Backfilled Area Surface Sediment Monitoring	1	LS	$ 25,000	$ 25,000		
DNR Lease	0.3	acre	$ 20,000	$ 6,000		Offshore cap area off property
Subtotal				*$ 151,000*		
Tax	9.5%		$ 151,000	$ 14,345		Sales Tax
Contingency	25%		$ 165,345	$ 41,336		
Total 1st Year O&M Cost				$ 206,681		
Annual O&M						
Groundwater Monitoring	1	LS	$ 25,000	$ 25,000	Project experience	20 wells annually
Upland Cap inspection	6	hour	$ 80	$ 480	labor estimate	
DNR Lease	0.3	acre	$ 20,000	$ 6,000		Offshore cap area off property
Subtotal				*$ 31,480*		
Tax	9.5%		$ 31,480	$ 2,991		Sales Tax
Contingency	25%		$ 34,471	$ 8,618		
Total Annual O&M Cost				$ 43,088		
Professional Services (as percent of Annual O&M costs)						
Project management/Reporting	10%		$ 43,088	$ 4,309		
Total, Annual O&M:				**$ 47,397**		
Total Estimated O&M, 100 Years, No NPV Analysis:				**$ 4,900,000**		
Periodic Costs						
Sand Cap and ENR						
Sediment Sand Cap and ENR Sampling at 2 years				$ 25,000		
Sediment Sand Cap and ENR Sampling at 5 years				$ 25,000		
Sediment Sand Cap and ENR Sampling at 10 years				$ 25,000		
Sediment Cap Inspection at 2 years				$ 15,000		
Sediment Cap Inspection at 5 years				$ 15,000		
Sediment Cap Inspection at 10 years				$ 15,000		
Sand Cap Shoreline Maintenance at 30 years				$ 25,000		
Sand Cap Shoreline Maintenance at 60 years				$ 25,000		
Sand Cap Shoreline Maintenance at 90 years				$ 25,000		
Subtotal				*$ 195,000*		
TOTAL ESTIMATED COST, NO NPV ANALSYS				**$ 264,495,000**		
Net Present Value Analysis						
Annual O&M	100	year	$ 47,397	$ 2,356,613		
1st year O&M	1	LS	$ 206,681	$ 206,681		
Sediment Sand Cap and ENR Sampling at 2 years	1	LS	$ 25,000	$ 24,219		
Sediment Sand Cap and ENR Sampling at 5 years	1	LS	$ 25,000	$ 23,093		
Sediment Sand Cap and ENR Sampling at 10 years	1	LS	$ 25,000	$ 21,331		
Sediment Cap Inspection at 2 years	1	LS	$ 15,000	$ 14,531		
Sediment Cap Inspection at 5 years	1	LS	$ 15,000	$ 13,856		
Sediment Cap Inspection at 10 years	1	LS	$ 15,000	$ 12,798		
Sand Cap Shoreline Maintenance at 30 years	1	LS	$ 25,000	$ 15,528		
Sand Cap Shoreline Maintenance at 60 years	1	LS	$ 25,000	$ 9,645		
Sand Cap Shoreline Maintenance at 90 years	1	LS	$ 25,000	$ 5,991		
2013 discount rate for NPV	1.6%					
Total Estimated O&M and Periodic NPV				**$ 2,704,286**		
TOTAL ESTIMATED COST				**$ 262,104,286**		

Notes:
1. Mobilization/Demobilization costs are assumed to include equipment transport and setup, temporary erosion and sedimentation control (TESC) measures, bonds, and insurance.
2. Contingency costs include miscellaneous costs not currently itemized due to the current (preliminary) stage of design development, as well as costs to address unanticipated conditions encountered during construction.

Table D-10 - Alternative 10 Cost Estimate

Quendall Terminals
Renton, Washington

Site:	Quendall Terminals			
Remedial Action Description:	Alternative	10	Containment with Removal of Contaminated Soil and Removal of Contaminated Sediment	
Cost Estimate Accuracy:	FS Screening Level (+50/-30 percent)			
Key Assumptions and Quantities:	Capping of Upland Soil			
(see Appendix E for calculations)	21.6	acre	total area	
	940,896	SF	total area	
	133,521	SF	permeable area along shoreline	
	5,023	BCY	habitat excavation overlap	
	104,544	BCY	total volume	based on 3' cap thickness
	Enhanced Natural Recovery - Sand Material			
	14,300	BCY	total volume	
	Engineered Sand Cap			
	9,700	BCY	total sand volume	
	800	BCY	removal volume for offsetting sand cap	
	15,000	SF	area for offsetting sand cap	
	0.3	acre	DNR lease area	
	Soil/Sediment Density			
	1.6	tons/BCY	soil density	
	1.3	tons/BCY	sediment density	
	0.7	tons/CY	organoclay density	
	Removal of Upland Source Area Soil			
	705,400	BCY	total volume	
	14.1	acre	total area	
	Volume of sediment removal			
	172,300	BCY	sediment removal	
	173,100	BCY	total sediment removal volume (including for offsetting cap)	
	148,600	BCY	mechanical dredging	
	23,700	BCY	hydraulic dredging	
	1,170	BCY	residual cover - organoclay	
	5,400	BCY	residual cover - sand	
	165,900	BCY	backfill	
	91,860	SF	sheet pile area	
	Dewatering to maintain wet removal for upland soil			
	281	gpm	maximum upland dewatering rate	
	221	gpm	average upland dewatering rate	
	60	each	deep aquifer depressurization wells	
	6.78	year	upland soil removal time	
	31	feet	average excavation depth	
	65	feet	min.embed. depth	
	399,505	SF	shoring wall area	
	353,331	SF	extra embedment	
	Pump-and-Treat of remaining contaminated groundwater			
	8	wells		
	90	gpm	total	

Item	Quantity	Unit	Unit Cost	Total Cost	Source	Notes
CAPITAL CONSTRUCTION COSTS						
Upland Soil Cap						
Mobilization/Demobilization	1	LS	$ 268,073	$ 268,073	percentage of construction costs	includes temporary facilities for duration of construction
Site Preparation	22	acre	$ 6,900	$ 149,040	Costworks	clearing, grubbing brush and stumps
Geotextile marker layer	104,544	SY	$ 2	$ 158,907	Costworks	non-woven, 120lb tensile strength
Import Fill - Permeable Cap	104,544	BCY	$ 30	$ 3,136,320	project experience	
Compaction	104,544	BCY	$ 5	$ 522,720	project experience	
Habitat Area - excavation	5,023	BCY	$ 6	$ 30,139		
Habitat Area - non-hazardous transport and disposal	8,037	ton	$ 50	$ 401,858		
Hydroseeding	14,836	SY	$ 1	$ 8,901	Costworks	includes seed and fertilizer for wetland area
Stormwater collection and detention system	1,500	LF	$ 40	$ 60,000	project experience	media filter drain
Subtotal				*$ 4,735,959*		
Tax	9.5%		$ 4,735,959	$ 449,916		Sales Tax
Contingency	25%		$ 5,185,875	$ 1,296,469		
Total Upland Soil Cap Cost				$ 6,482,343		
Enhanced Natural Recovery						
Mobilization/Demobilization	1	LS	$ 49,248	$ 49,248		
Sand Material	22,880	ton	$ 20	$ 457,600	vendor quote	
Sand Placement	22,880	ton	$ 15	$ 343,200	project experience	ENR placed as one lift
Confirmation of Placement	1	LS	$ 20,000	$ 20,000		
Subtotal				*$ 870,048*		
Tax	9.5%		$ 870,048	$ 82,655		Sales Tax
Contingency	25%		$ 952,703	$ 238,175.64		
Total Enhanced Natural Recovery Cost				$ 1,190,878		
Engineered Sand Cap						
Mobilization/Demobilization	1	LS	$ 38,898	$ 38,898		
Sand Material	15,520	ton	$ 20	$ 310,400	vendor quote	
Sand Placement	15,520	ton	$ 20	$ 310,400	project experience	Sand Cap placed in multiple lifts
Geotextile Separation Layer	15,000	SF	$ 1	$ 7,500	Vendor quote	Only in nearshore area
Confirmation of Placement	1	LS	$ 20,000	$ 20,000		
Subtotal				*$ 687,198*		
Tax	9.5%		$ 687,198	$ 65,284		Sales Tax
Contingency	25%		$ 752,482	$ 188,120		
Total Engineered Sand Cap Cost				$ 940,602		
Upland Soil Removal						
Mobilization/Demobilization	1	LS	$ 9,567,627	$ 9,567,627	percentage of construction costs	includes temporary facilities for duration of construction
Excavation	705,400	BCY	$ 6	$ 4,232,400	project experience	
Soil Handling and Stockpiling	705,400	BCY	$ 5	$ 3,527,000	project experience	segregation into hazardous/non-hazardous
Analytical Sampling	200	ea	$ 500	$ 100,000	project experience	VOCs and SVOCs
Compaction	705,400	BCY	$ 5	$ 3,527,000	project experience	
On-Site Treatment - Thermal Desorption	1,128,640	ton	$ 95	$ 107,220,800	vendor estimate	
Shoring	399,505	SF	$ 72	$ 28,764,360	project experience	sheet pile (see Appendix F)
Extra Embedment	353,331	SF	$ 15	$ 5,299,966	project experience	
Dewatering - Deep Aquifer Depressurization Wells and Pumps	60	ea	$ 40,000	$ 2,400,000	project experience	
Dewatering - Equalization Tank	82	month	$ 980	$ 80,360	project experience	Rental - 20,000 gallon tank
Dewatering - Treatment system	82	month	$ 8,066	$ 661,412	Vendor quote	rental system: DNAPL separation, air stripping, filtration, GAC vessels
Dewatering - Arsenic Treatment and Media	1	LS	$ 23,071	$ 23,071	Vendor quote	based on usage rate of 4% by weight
Dewatering - Carbon Replacement	2,476	day	$ 132	$ 327,144	Vendor quote	based on usage rate of 65 lb/day @ 50gpm - $0.46/lb
Dewatering - Carbon Disposal	356	ton	$ 400	$ 142,236	Vendor quote	
Dewatering - Coagulant	6,578	lb	$ 2	$ 14,800	Vendor quote	$2.25 per lb, 1mg/L concentration, average flow rate
Dewatering - Miscellaneous Equipment	20%		$ 5,590,767	$ 1,118,153	percentage of dewatering capital cos	
Dewatering - Equipment Operation and Maintenance	2,476	day	$ 700	$ 1,732,978	labor estimate	1 full-time operator, $70/hr, 10hr/day
Dewatering - Power	82	month	$ 2,540	$ 208,280	project experience	$0.0996/KWH estimated power rate
Dewatering - Outfall Piping	50	LF	$ 10	$ 486	Costworks	8" Concrete discharge pipe
Monitoring Well Installation	20	ea	$ 4,000	$ 80,000	project experience	confirmation monitoring program
Subtotal				*$ 169,028,073*		
Tax	9.5%		$ 169,028,073	$ 16,057,667		Sales Tax
Contingency	35%		$ 185,085,739	$ 64,780,009		
Total Upland Soil Removal Cost				$ 249,865,748		
Sediment Removal						
Mobilization/Demobilization	1	LS	$ 2,106,270	$ 2,106,270		
Mechanical Dredging	149,400	BCY	$ 35	$ 5,229,000		Mechanical dredging in nearshore and for offsetting nearshore cap
Hydraulic Dredging	23,700	BCY	$ 60	$ 1,422,000	Project experience	Assumes specialty hydraulic for T-Dock/Offshore
Debris Removal and Disposal	1	LS	$ 75,000	$ 75,000		Removal of piling
Transloading/Material Handling	173,100	BCY	$ 15	$ 2,596,500		
Dewatering	173,100	BCY	$ 10	$ 1,644,450	vendor quote	Assumes 5% amendment by weight
Water Treatment	1	LS	$ 500,000	$ 500,000	Project experience	
Residuals Cover Bulk Organoclay Material - (PM-199)	837	ton	$ 3,250	$ 2,720,689	Quote from Cetco	
Residuals Cover Sand Material	8,640	ton	$ 20	$ 172,800	vendor quote	
Residuals Cover Material Placement	9,477	ton	$ 15	$ 142,157	project experience	
Backfill Material	265,440	ton	$ 20	$ 5,308,800	vendor quote	
Backfill Material Placement	265,440	ton	$ 15	$ 3,981,600	project experience	Backfill placed in bulk
Transportation and Disposal - Non-Hazardous	225,030	ton	$ 50	$ 11,251,500		Subtitle D landfill disposal
Dredging Confirmation	1	LS	$ 60,000	$ 60,000		
Subtotal				*$ 37,210,766*		
Tax	9.5%		$ 37,210,766	$ 3,535,023		Sales Tax
Contingency	25%		$ 40,745,788	$ 10,186,447		
Total Sediment Removal Cost				$ 50,932,235		
Sheet Pile Enclosure						
Mobilization/Demobilization	1	LS	$ 496,044	$ 496,044	Project experience	
Steel Unit Cost	91,860	SF	$ 35	$ 3,215,100	Project experience	
Installation Unit Cost	91,860	SF	$ 45	$ 4,133,700	Project experience	
Removal Unit Cost	91,860	SF	$ 15	$ 1,377,900	Project experience	
Salvage Unit Value	4,593,000	lb	$ (0.1)	$ (459,300)	Project experience	50 pounds per sf
Subtotal				*$ 8,763,444*		
Tax	9.5%		$ 8,763,444	$ 832,527		Sales Tax
Contingency	25%		$ 9,595,971	$ 2,398,993		
Total Sheet Pile Enclosure Cost				$ 11,994,964		
Sediment Environmental Controls and Monitoring						
Water Quality Monitoring	250	day	$ 2,500	$ 625,000		
Water Quality Controls and BMPs (Absorbent Booms, Silt Curtains, Oil Boor	1	LS	$ 200,000	$ 200,000		
Odor Control	220	day	$ 2,500	$ 550,000		
Noise Monitoring	1	LS	$ 30,000	$ 30,000		
Erosion Protection for Shoreline Area	1	LS	$ 250,000	$ 250,000		
Subtotal				*$ 1,655,000*		
Tax	9.5%		$ 1,655,000	$ 157,225		Sales Tax
Contingency	25%		$ 1,812,225	$ 453,056		
Total Sediment Environmental Controls and Monitoring Cost				$ 2,265,281		

Table D-10 - Alternative 10 Cost Estimate

DRAFT FINAL

Quendall Terminals
Renton, Washington

Pump and Treat Installation						
Treatment System	1	LS	$ 154,000	$ 154,000	Vendor quote	DNAPL separation, air stripping, filtration, GAC vessels
Arsenic Treatment and Media	1	LS	$ 23,071	$ 23,071	Vendor quote	Two - 3000lb vessels
Deep Aquifer Wells and Pumps	6	each	$ 40,000	$ 240,000		
Piping/Trenching	1,400	LF	$ 8	$ 11,760	Costworks	Includes electrical conduit and water transfer piping
Treatment Enclosure	500	SF	$ 189	$ 94,500	Costworks	1-Story w/office on metal studs
Power Installation	1	LS	$ 8,500	$ 8,500	Project experience	
Miscellaneous Items and Infrastructure	50%		$ 531,831	$ 265,916		percentage of pump and treat capital costs
Instrumentation and Automated Controls	10%		$ 531,831	$ 53,183		percentage of pump and treat capital costs
Subtotal				*$ 850,930*		
Tax	9.5%		$ 850,930	$ 80,838		Sales Tax
Contingency	25%		$ 931,768	$ 232,942		
Total Pump and Treat Installation Cost				$ 1,164,710		
Subtotal Construction Costs				**$ 324,836,762**		
Professional Services (as percent of construction and contingency costs)						
Project management	5%		$ 324,836,762	$ 16,241,838		
Remedial design	6%		$ 324,836,762	$ 19,490,206		Includes treatability studies for remedy components as necessary
Construction management	6%		$ 324,836,762	$ 19,490,206		
Subtotal				*$ 55,222,250*		
Total Estimated Capital Cost				**$ 380,100,000**		
O&M COSTS						
1st Year O&M						
GW Monitoring	1	LS	$ 80,000	$ 80,000	Project experience	
Sediment Sand Cap and ENR Sampling	1	LS	$ 25,000	$ 25,000	Project experience	
Sediment Cap Inspection	1	LS	$ 15,000	$ 15,000	Project experience	Visual and In-Water (Bathymetric/ Sediment Profile Image)
Backfilled Area Surface Sediment Monitoring	1	LS	$ 25,000	$ 25,000		
DNR Lease	0.3	acre	$ 20,000	$ 6,000		Offshore cap area off property
Subtotal				*$ 151,000*		
Tax	9.5%		$ 151,000	$ 14,345		Sales Tax
Contingency	25%		$ 165,345	$ 41,336		
Total 1st Year O&M Cost				$ 206,681		
Annual O&M						
Groundwater Monitoring	1	LS	$ 25,000	$ 25,000	Project experience	20 wells annually
Upland Cap inspection	6	hour	$ 80	$ 480	labor estimate	
DNR Lease	0.3	acre	$ 20,000	$ 6,000		Offshore cap area off property
Pump and Treat Maintenace	20%	capital	$ 850,930	$ 170,186		20% of capital cost
Pump and Treat GAC Replacement/Displosal	1.2	ton	$ 1,320	$ 1,566		Based on 6.5lb/day usage rate
Pump and Treat Coagulant	395	lb	$ 2	$ 889	Vendor quote	$2.25 per lb, 1mg/L concentration
Pump and Treat Power Consumption	12	month	$ 1,140	$ 13,680	Project experience	
Pump and Treat Monitoring and Reporting	2,080	hour	$ 70	$ 145,600		40 hours per week
Subtotal				*$ 363,400*		
Tax	9.5%		$ 363,400	$ 34,523		Sales Tax
Contingency	25%		$ 397,924	$ 99,481		
Total Annual O&M Cost				$ 497,404		
Professional Services (as percent of Annual O&M costs)						
Project management/Reporting	10%		$ 497,404	$ 49,740		
Total, Annual O&M:				**$ 547,145**		
Total Estimated O&M, 100 Years, No NPV Analysis:				**$ 54,900,000**		
Periodic Costs						
Sand Cap and ENR						
Sediment Sand Cap and ENR Sampling at 2 years				$ 25,000		
Sediment Sand Cap and ENR Sampling at 5 years				$ 25,000		
Sediment Sand Cap and ENR Sampling at 10 years				$ 25,000		
Sediment Cap Inspection at 2 years				$ 15,000		
Sediment Cap Inspection at 5 years				$ 15,000		
Sediment Cap Inspection at 10 years				$ 15,000		
Sand Cap Shoreline Maintenance at 30 years				$ 25,000		
Sand Cap Shoreline Maintenance at 60 years				$ 25,000		
Sand Cap Shoreline Maintenance at 90 years				$ 25,000		
Pump and treat system						
Replace P&T System at 20 yrs				$ 850,930		
Replace P&T System at 40 yrs				$ 850,930		
Replace P&T System at 60 yrs				$ 850,930		
Replace P&T System at 80 yrs				$ 850,930		
Subtotal				*$ 3,598,718*		
TOTAL ESTIMATED COST, NO NPV ANALSYS				**$ 438,598,718**		
Net Present Value Analysis						
Annual O&M	100	year	$ 547,145	$ 27,204,391		
1st year O&M	1	LS	$ 206,681	$ 206,681		
Sediment Sand Cap and ENR Sampling at 2 years	1	LS	$ 25,000	$ 24,219		
Sediment Sand Cap and ENR Sampling at 5 years	1	LS	$ 25,000	$ 23,093		
Sediment Sand Cap and ENR Sampling at 10 years	1	LS	$ 25,000	$ 21,331		
Sediment Cap Inspection at 2 years	1	LS	$ 15,000	$ 14,531		
Sediment Cap Inspection at 5 years	1	LS	$ 15,000	$ 13,856		
Sediment Cap Inspection at 10 years	1	LS	$ 15,000	$ 12,798		
Sand Cap Shoreline Maintenance at 30 years	1	LS	$ 25,000	$ 15,528		
Sand Cap Shoreline Maintenance at 60 years	1	LS	$ 25,000	$ 9,645		
Sand Cap Shoreline Maintenance at 90 years	1	LS	$ 25,000	$ 5,991		
Replace P&T System at 20 yrs	1	LS	$ 850,930	$ 619,469		
Replace P&T System at 40 yrs	1	LS	$ 850,930	$ 450,968		
Replace P&T System at 60 yrs	1	LS	$ 850,930	$ 328,300		
Replace P&T System at 80 yrs	1	LS	$ 850,930	$ 239,000		
2013 discount rate for NPV	1.6%					
Total Estimated O&M and Periodic NPV				**$ 29,189,800**		
TOTAL ESTIMATED COST				**$ 409,289,800**		

Notes:
1. Mobilization/Demobilization costs are assumed to include equipment transport and setup, temporary erosion and sedimentation control (TESC) measures, bonds, and insurance.
2. Contingency costs include miscellaneous costs not currently itemized due to the current (preliminary) stage of design development, as well as costs to address unanticipated conditions encountered during construction.

APPENDIX E

Engineering Calculation Sheets

Table of Contents: Engineering Calculation Sheets

Engineering Calculation Sheet E-1: Habitat Excavation Volumes

Site:	Quendall Terminals		**Engineer**	**Date**
Calculations:	Estimate overexcavation volume to place clean cap in habitat area	Calculations By:	ELG	8/7/2013
		Checked By:	JJP	8/14/2013

Assumptions:

- Existing grade within future habitat area maintained
- 3-foot-depth 100 feet inland along shoreline
- Exclude alternative-specific DNAPL excavation areas ("Overlap Area")

Equations:

Habitat Excavation Area = Total Area - Overlap Area

Excavation Volume = Depth x Excavation Area

Alternative	Total Habitat Area in Square Feet[1]	Area of Excavation Overlap in Square Feet[2]	Area of Habitat for Excavation in Square Feet	Depth of Excavation in Feet	Volume of Non-Hazardous Soil Excavation in BCY
2,3,5,7	133,521	--	133,521	3	14,836
4,6	133,521	21,556	111,965	3	12,441
8	133,521	46,035	87,486	3	9,721
9-10	133,521	88,312	45,209	3	5,023

Notes:

[1] Area based on AutoCad calculation for 'Permeable Cap/Habitat Area' on Figure 6-1.

[2] Areas based on AutoCad calculation of excavation areas on Figures 6-6, 6-11, 6-16, 6-18, and 6-21 within 100 feet of the shoreline

Conversion factors:

1 cy = 27 CF

Engineering Calculation Sheet E-2: PRB and DNAPL Collection Trench Excavation Volumes

Site:	Quendall Terminals		**Engineer**	**Date**
Calculations:	Estimate the volume of hazardous and non-hazardous soil to be removed for PRB (Alternatives 3 through 6) and DNAPL collection trenches (Alternatives 3 and 4)	Calculations By:	ELG	8/7/2013
		Checked By:	JJP	8/14/2013

Assumptions:

18% of soil removed contains DNAPL[(1)]
Soil containing DNAPL would be designated as hazardous waste

Equations:
Volume = Length x Width x Depth
Hazardous Soil Volume = 18% x Excavated Soil Volume
Non-Hazardous Soil Volume = Excavated Soil Volume - Hazardous Soil Volume

Trench	Depth in Feet	Width in Feet	Total Length in Feet	Excavated Soil Volume in BCY	Hazardous Soil Volume in BCY	Non-Hazardous Soil Volume in BCY
PRB	25	2	1,100	2,037	367	1,670
DNAPL Collection	25	2	500	926	167	759
Total:				**2,963**	**533**	**2,430**

Notes:
(1) Based on site-wide ratio of DNAPL-containing soil volume to DNAPL-containing soil and overburden soil volume (see Table G-6 of the RI Report).

Conversion factors:
1 cy = 27 CF

Engineering Calculation Sheet E-3: Alternatives 4 and 6 - Excavation Volumes

Site:	Quendall Terminals		**Engineer**	**Date**
Calculations:	Estimate the volume of upland soil to be removed under Alternatives 4 and 6	Calculations By:	ELG	8/7/2013
	Estimate area and perimeter of shoring walls under Alternatives 4 and 6	Checked By:	JJP	8/14/2013
	Estimate volume of hazardous and non-hazardous soil removed			

Assumptions:

18% of excavated soil contains DNAPL and would be designated as hazardous waste [(1)]

Equations:

Excavated Soil Volume = Area x Average DNAPL Depth

Exposed Shoring Wall Area = Perimeter x Average DNAPL Depth

Volume of Solidified DNAPL (gal) = soil volume (yd^3) x 1.6 $tons/yd^3$ × 909 kg/ton × 0.011 $kg_{BTEX+PAHs}/kg_{soil}$ × 3.05 $kg_{hydrocarbons}/kg_{BTEX+PAHs}$ × 264 gal/m^3 ÷ 1,040 $kg_{hydrocarbons}/m^3$

Excavation Area	Area in Square Feet	Area in Acres	Perimeter Length in Feet	Maximum DNAPL Depth in Feet	Average DNAPL Depth in Feet	Exposed Area of Shoring Wall in Square Feet	Volume of Soil to be Excavated in BCY	Hazardous Soil Volume in BCY	Estimated Volume of DNAPL Removed in Gallons	Non-Hazardous Soil Volume in BCY
QP-U DNAPL Area	21,556	0.5	636	19.0	15.9	10,109	12,700	2,286	28,315	10,414

Notes:

[(1)]Based on site-wide ratio of DNAPL-containing soil volume to DNAPL-containing soil and overburden soil volume (see Table G-6 of the RI Report).

Cell area and perimeter calculated by AutoCad based on excavation extent shown on Figure 6-6 and 6-11.

Average depth calculated using depth and area of thiessen polygons (See Appendix G of the RI Report) for borings within Excavation Area - see calculation sheet E-18.

Conversion factors:

1 acre = 43,560 SF

1 cy = 27 CF

Engineering Calculation Sheet E-4: Alternative 8 - Excavation Volumes

Site:	Quendall Terminals		**Engineer**	**Date**
Calculations:	Estimate the volume of upland soil to be removed under Alternative 8	Calculations By:	ELG	8/7/2013
	Estimate area and perimeter of shoring walls under Alternative 8	Checked By:	JJP	8/14/2013

Equations: Excavation Volume = Area x Average DNAPL Depth
Exposed Area = Perimeter x Average DNAPL Depth

Excavation Cell	Area in Square Feet[1]	Area in Acres	Perimeter Length in Feet	Maximum DNAPL Depth in Feet	Average DNAPL Depth in Feet	Exposed Area of Shoring Wall in Square Feet	Volume of Soil to be Excavated in BCY
1	15,672	0.4	502	33.7	25.5	12,804	14,800
2	10,105	0.2	447	22.0	20.6	9,210	7,700
3	164,325	3.8	1,626	13.8	9.1	14,782	55,300
4	86,433	2.0	1,752	17.8	14.2	24,913	45,500
5	12,616	0.3	471	24.0	24.0	11,304	11,200
6	5,773	0.1	321	26.5	26.5	8,507	5,700
7	74,327	1.7	1,319	22.0	14.1	18,603	38,800
8	14,529	0.3	488	19.0	16.6	8,122	9,000
9	24,276	0.6	778	15.0	11.7	9,113	10,500
10	12,809	0.3	426	31.5	24.5	10,451	11,600
TOTAL	**420,865**	**9.7**	**8,130**			**127,809**	**210,100**

Notes:
(1)Cell areas and perimeters calculated by AutoCad based on cells shown on Figure 6-16.
(2)Average depth calculated using depth and area of thiessen polygons (see Appendix G of the RI Report) for borings within each Excavation Cell - see calculation sheet E-19.

Conversion factors:
1 acre = 43,560 SF
1 cy = 27 CF

Engineering Calculation Sheet E-5: Alternative 9 - Excavation and Solidification Volumes

			Engineer	Date
Site:	Quendall Terminals			
Calculation:	Estimate the volume of upland soil to be removed and solidified under Alternative 9	Calculations By:	ELG	8/7/2013
	Estimate area and perimeter of shoring walls under Alternative 9	Checked By:	JJP	8/14/2013

Assumptions:

Area to be excavated includes:

Shallow Alluvium within benzo[a]pyrene and arsenic plume to a depth of 15 feet

Area to be solidified extends to same depth of excavation in Alternative 10

705,400 BCY total volume excavated for Alternative 10 - see calculation sheet E-6

Area of 4-foot-diameter auger solidification equal to area of Alternative 10 excavation cells penetrating the Deeper Alluvium

Procedure: Estimate the volume of each excavation cell and sum result

Subtract from total volume of upland soil to be treated to get volume solidified

Volume of Upland Soil Removed

Equations: Volume = Area x Depth

Exposed Area = Perimeter x Depth

Cell Number	Area in Square Feet	Perimeter in Feet	Depth in Feet	Volume in BCY	Exposed Sheet Pile Area in SF
1	177,498	1,901	15	98,600	28,515
2	137,990	1,662	15	76,700	24,930
3	140,036	1,574	15	77,800	23,610
4	160,980	1,622	15	89,400	24,330
Total	**616,504**	**6,759**		**342,500**	**101,385**

Volume of Upland Soil Solidified

Equations: Volume of Upland Soil Solidified = Volume of Upland Soil Removed for Alternative 10 - Volume of Upland Soil Removed for Alternative 9

362,900 BCY

Volume of Upland Soil Solidified with 4-foot-diameter auger = 25 feet x Area of Cells 2 and 12 from calculation sheet E-6

76,999 BCY

Notes:

Cell areas and perimeters calculated by AutoCad based on cells shown on Figure 6-18

Conversion factors:

1 acre = 43,560 SF

1 cy = 27 CF

Engineering Calculation SheetE-6: Alternative 10 - Excavation Volumes

Site:	Quendall Terminals		**Engineer**	**Date**
Calculations:	Estimate the volume of upland soil to be removed under Alternative 10	Calculations By:	ELG	8/7/2013
	Estimate area and perimeter of shoring walls under Alternative 10	Checked By:	JJP	8/14/2013

Assumptions:

Area includes:

Shallow Alluvium within benzo[a]pyrene and arsenic plume

Deeper Alluvium includes benzo[a]pyrene plume

Equations: Volume = Area x Depth

Exposed Area = Perimeter x Depth

Cell Number	Area in Square Feet[1]	Perimeter in Feet	Depth in Feet	Depth Basis	Volume in BCY	Exposed Shoring Wall Area in Square Feet
1	38,499	775	25	(2)	35,600	19,375
2	36,768	801	40	(4)	54,500	32,040
3	40,078	801	35	(3)	52,000	28,035
4	45,320	895	35	(3)	58,700	31,325
5	47,719	874	25	(2)	44,200	21,850
6	40,456	824	35	(3)	52,400	28,840
7	30,174	701	35	(3)	39,100	24,535
8	29,388	820	25	(2)	27,200	20,500
9	53,560	943	25	(2)	49,600	23,575
10	29,539	690	35	(3)	38,300	24,150
11	32,969	745	35	(3)	42,700	26,075
12	46,391	862	40	(4)	68,700	34,480
13	46,504	910	25	(2)	43,100	22,750
14	31,043	728	25	(2)	28,700	18,200
15	25,384	665	35	(3)	32,900	23,275
16	40,740	820	25	(2)	37,700	20,500
Total	**614,532**	**12,854**			**705,400**	**399,505**
Total Area	**14**	**Acres**				
Avg. Depth	**31**	**Feet**				

Notes:

[1] Cell areas and perimeters calculated by AutoCad based on cells shown on Figure 6-21

[2] 25-foot depth assumes average depth of B[a]P contamination in areas (other than deeper DNAPL at MC-1 and BH-30) without arsenic exceedences.

[3] 35-foot depth assumes average depth to Shallow Alluvium in areas of elevated arsenic concentrations in groundwater.
Applied to cells, except those covered by Note 4, where the shallow arsenic plume is estimated to cover more than 50% of cell area.

[4] 40-foot depth assumes B[a]P contamination in Deeper Alluvium extends on average 5 feet into Deeper Alluvium in cells with BH-30 (cell 2) and MC-1 (cell 12).

Conversion factors:
1 acre = 43,560 SF
1 cy = 27 CF

Engineering Calculation Sheet E-7: DNAPL Volume Calculations

Site:	Quendall Terminals		**Engineer**	**Date**
Calculations:	Estimate the volume of DNAPL using depth and area of theissen polygons	Calculations By:	ELG	8/7/2013
		Checked By:	JJP	8/14/2013

Equations: Volume of DNAPL Contaminated Soil = DNAPL Thickness x Area

Volume of Excavated DNAPL (gal) = soil volume (yd^3) x 1.6 tons/yd^3 × 909 kg/ton × 0.011 $kg_{BTEX+PAHs}/kg_{soil}$ × 3.05 $kg_{hydrocarbons}/kg_{BTEX+PAHs}$ × 264 gal/m^3 ÷ 1,040 $kg_{hydrocarbons}/m^3$

(see Appendix G of the RI Report)

Boring Containing DNAPL	Site DNAPL Area	Total DNAPL Thickness in Feet	DNAPL Thickness to 20' Below Ground Surface	DNAPL Thickness to 15' Below Ground Surface	Maximum Depth of DNAPL in Feet	Area in Square Feet	Volume of DNAPL-Contaminated Soil or Sediment in Cubic Feet	Volume of Soil or Sediment to Bottom of DNAPL in Cubic Feet	Volume of DNAPL-Contaminated Soil to 15' Below Ground Surface in Cubic Feet
BH-5	Quendall Pond/North Sump Area	2	2	1	19	5,879	11,758	111,701	218
BH-20C	Quendall Pond/North Sump Area	1	0	0	26.5	5,542	5,542	146,863	-
BH-23	Quendall Pond/North Sump Area	5.5	4.5	3	24	9,113	50,121	218,710	1,013
BH-5B	Quendall Pond/North Sump Area	2.5	2.5	0.5	16	3,076	7,690	49,215	57
HC-2	Quendall Pond/North Sump Area	3.9	3.9	3.8	15.1	14,230	55,498	214,875	2,003
QP-1[3]	Quendall Pond/North Sump Area	2	2	0	18.5	4,649	9,297	85,997	-
QP-5	Quendall Pond/North Sump Area	1	1	1	12	4,210	4,210	50,520	156
RB-9	Quendall Pond/North Sump Area	5.2	5	0	20.2	6,694	34,811	135,226	-
RB-11	Quendall Pond/North Sump Area	2	2	0	18	2,810	5,620	50,579	-
RB-12	Quendall Pond/North Sump Area	0.4	0.4	0	18	4,639	1,856	83,511	-
RB-14	Quendall Pond/North Sump Area	2	1	0	21	3,800	7,601	79,809	-
RB-19	Quendall Pond/North Sump Area	1.6	1.6	1.6	12.6	7,274	11,638	91,647	431
RB-23	Quendall Pond/North Sump Area	4	4	4	12	6,539	26,156	78,469	969
SP-2[3]	Quendall Pond/North Sump Area	0.2	0	0	22	2,583	517	56,826	-
SP-3[3]	Quendall Pond/North Sump Area	2	2	1	16	5,073	10,145	81,164	188
SP-4[3]	Quendall Pond/North Sump Area	1.9	1.9	1.9	12.5	3,412	6,483	42,648	240
SP-5	Quendall Pond/North Sump Area	6	6	5	16	7,037	42,221	112,590	1,303
SP-6	Quendall Pond/North Sump Area	3.5	3.5	3.5	13	9,418	32,961	122,428	1,221
SP-7	Quendall Pond/North Sump Area	3.2	3.2	3	17.8	9,810	31,393	174,621	1,090
SP-8[3]	Quendall Pond/North Sump Area	2.2	2.2	0.8	18	1,668	3,669	30,015	49
SWB-4	Quendall Pond/North Sump Area	1.5	1.5	1.5	14	1,619	2,429	22,667	90
SWB-4A	Quendall Pond/North Sump Area	1	1	1	11	6,404	6,404	70,440	237
						Total DNAPL-Containing Soil in Cubic Yards	13,630		
							Total Soil Volume to Bottom of DNAPL in Cubic Yards	78,167	
								Total Volume of DNAPL to 15' in Gallons	114,750
								Total Volume of DNAPL in Gallons	168,831
BH-21A	Former May Creek Channel Area	5.5	5.5	1.5	19	4,773	26,252	90,687	265
BH-30C	Former May Creek Channel Area	3.25	2.75	2.75	33.75	3,558	11,564	120,087	362
HC-7	Former May Creek Channel Area	6.5	6.5	6.5	15	5,455	35,458	81,827	1,313
MC-1	Former May Creek Channel Area	8.75	6.75	4.25	31.5	3,840	33,603	120,970	604
MC-2	Former May Creek Channel Area	1.4	1.4	1.4	14.5	3,755	5,257	54,451	195
MC-7	Former May Creek Channel Area	2	2	0	18	2,389	4,778	43,000	0
MC-8	Former May Creek Channel Area	3	3	3	14	1,546	4,639	21,647	172
MC-13	Former May Creek Channel Area	0.3	0.3	0	18.3	4,291	1,287	78,523	0
MC-16[3]	Former May Creek Channel Area	0.2	0.2	0.2	13	1,428	286	18,564	11
MC-18	Former May Creek Channel Area	0.8	0.8	0.8	13	12,477	9,982	162,202	370
MC-20	Former May Creek Channel Area	2.5	2.5	2.5	12.25	9,527	23,818	116,709	882
MC-23	Former May Creek Channel Area	2.5	2.5	2.5	13	6,507	16,266	84,586	602
Q2-D	Former May Creek Channel Area (portion) [2]	11	7	2.5	30	1,236	13,596	37,080	114
Q4	Former May Creek Channel Area (portion) [2]	2.5	2.5	1	16.5	870	2,175	14,355	32
SP-1	Former May Creek Channel Area	0.6	0.6	0.6	9.8	2,699	1,619	26,450	60
						Total DNAPL-Containing Soil/Sediment in Cubic Yards	7,059		
							Total Soil Volume to Bottom of DNAPL in Cubic Yards	39,672	
								Total Volume of DNAPL to 15' in Gallons	61,726
								Total Volume of DNAPL in Gallons	87,430
BH-8	Still House Area	4	4	4	12.5	18,456	73,825	230,704	2,734
BH-9	Still House Area	2	2	2	3.5	21,173	42,345	74,104	1,568
HC-4	Still House Area	1	1	1	10	20,752	20,752	207,520	769
HC-5	Still House Area	2.5	2.5	2.5	13	5,429	13,573	70,578	503
Q1-D	Still House Area (portion) [2]	6	4	2	22	4,139	24,834	91,058	307
Q7	Still House Area (portion) [2]	0.5	0.5	0	19	3,018	1,509	57,342	-
QP-6	Still House Area	1.25	1.25	1.25	12.25	9,872	12,340	120,933	457
QP-7	Still House Area	2.25	2.25	2.25	13.75	13,112	29,502	180,290	1,093
						Total DNAPL-Containing Soil in Cubic Yards	8,099		
							Total Soil Volume to Bottom of DNAPL in Cubic Yards	38,242	
								Total Volume of DNAPL to 15' in Gallons	92,034
								Total Volume of DNAPL in Gallons	100,321
Q1-D	Railroad Loading Area (portion) [2]	6	4	2	22	1,357	8,142	29,854	101
Q2-C	Railroad Loading Area	1	1	0	18	1,868	1,868	33,626	-
Q2-D	Railroad Loading Area (portion) [2]	11	7	2.5	30	598	6,578	17,940	55
Q4	Railroad Loading Area (portion) [2]	2.5	2.5	1	16.5	1,566	3,915	25,839	58
Q7	Railroad Loading Area (portion) [2]	0.5	0.5	0	19	1,758	879	33,402	-
Q9	Railroad Loading Area	8.5	6	1.5	25	2,839	24,132	70,975	158
						Total DNAPL-Containing Soil in Cubic Yards	1,686		
							Total Soil Volume to Bottom of DNAPL in Cubic Yards	7,838	
								Total Volume of DNAPL to 15' in Gallons	4,603
								Total Volume of DNAPL in Gallons	20,880
QPN-07 [1]	Nearshore Quendall Pond Area (DA-8)	0.2	--	--	8.7	3,971	794	34,548	--
VS2[1]	Nearshore Quendall Pond Area (DA-8)	0.3	--	--	16.3	17,057	5,117	278,029	--
QPN-02 [1]	Nearshore Quendall Pond Area (DA-6)	1.7	--	--	7.4	5,035	8,560	37,259	--
VS30[1]	Nearshore Quendall Pond Area (DA-6)	5	--	--	9	7,460	37,300	67,140	--
NS15-C1 [1]	Nearshore Quendall Pond Area (DA-8)	0.1	--	--	9.3	4,235	424	39,389	--
SP-2[3]	Nearshore Quendall Pond Area (DA-8)	0.2	--	--	22	10,332	2,066	227,304	--
SP-3[3]	Nearshore Quendall Pond Area (DA-6)	2	--	--	16	1,268	2,536	20,291	--
SP-4[3]	Nearshore Quendall Pond Area (DA-6)	1.9	--	--	12.5	2,275	4,322	28,432	--
SP-8[3]	Nearshore Quendall Pond Area (DA-6)	2.2	--	--	18	1,668	3,669	30,015	--
QP-1[3]	Nearshore Quendall Pond Area (DA-8)	2	--	--	18.5	1,550	3,099	28,666	--
						Total DNAPL-Containing Sediment in Cubic Yards	2,514		
							Total Sediment Volume to Bottom of DNAPL in Cubic Yards	29,299	
								Total Volume of DNAPL in Gallons	31,143
EPA-1	T-Dock Area (DA-2)	0.5	--	--	0.5	8,048	4,024	4,024	--
EPA-8	T-Dock Area (DA-5)	1	--	--	1	2,050	2,050	2,050	--
TD-01	T-Dock Area (DA-4)	0.1	--	--	0.8	16,096	1,610	12,877	--
TD-08	T-Dock Area (DA-2)	0.1	--	--	0.4	10,123	1,012	4,049	--
VS-27	T-Dock Area (DA-3)	0.2	--	--	2.7	10,592	2,118	28,598	--
VT-1	T-Dock Area (DA-1)	1	--	--	1	11,251	11,251	11,251	--
VT-4	T-Dock Area (DA-1)	3.8	--	--	3.8	15,057	57,217	57,217	--
						Total DNAPL-Containing Sediment in Cubic Yards	2,936		
							Total Sediment Volume to Bottom of DNAPL in Cubic Yards	4,447	
								Total Volume of DNAPL in Gallons	36,371
MC-16[3]	Former May Creek Channel Area (DA-7)	0.2	--	--	13	1,428	286	18,564	--
						Total DNAPL-Containing Sediment in Cubic Yards	11		
							Total Sediment Volume to Bottom of DNAPL in Cubic Yards	688	
								Total Volume of DNAPL in Gallons	131
						SOIL TOTAL IN CUBIC YARDS	**30,474**		
						SEDIMENT TOTAL IN CUBIC YARDS	**5,451**		
						SOIL TOTAL IN CUBIC YARDS		**163,919**	
						SEDIMENT TOTAL IN CUBIC YARDS		**34,433**	
						SOIL DNAPL TOTAL IN GALLONS			**377,462**
						SOIL DNAPL TOTAL TO 15' IN GALLONS			**273,113**
						SEDIMENT DNAPL TOTAL IN GALLONS			**67,646**

Notes:

-- Not calculated

Calculation sheet adapted from Table G-5 of the RI Report

See Tables G-1 through G-4 of the RI Report for DNAPL depth intervals at each boring.

See Figure G-1 of the RI Report for Thiessen polygon locations associated with each boring.

[1] Sediment boring in the offshore portion of the Quendall Pond/North Sump Area. The volumes shown in columns 6 and 7 are of sediment rather than soil.

[2] Includes area in both the Former May Creek Channel Area or the Still House Area (on Quendall property) and the Railroad Loading Area (on Railroad property), as follows:

Boring	Area of Theissen Polygon in Sq. Feet: Quendall Property	Railroad Property	Total Area in Square Feet
BH-17B	10,565	5,096	15,661
HC-8	4,074	1,749	5,823
MC-24	7,477	839	8,316
Q13	1,696	1,426	3,122
Q14	14,141	9,752	23,893
Q17	1,141	9,019	10,160
Q1-D	4,139	1,357	5,496
Q2-D	1,236	598	1,834
Q4	870	1,566	2,436
Q5	5,023	2,232	7,255
Q6	4,694	1,983	6,677
Q7	3,018	1,758	4,776

[3] Includes area in both upland soil and nearshore sediment, as follows:

Theissen Polygons split along shoreline: Boring	Estimated Percent Upland	Total Area of Theissen Polygon in sq. Feet	Portion of Theissen Polygon in Sq. Feet: Upland Soil	Nearshore Sediment
SP-2	20%	12,915	2,583	10,332
SP-3	80%	6,341	5,073	1,268
SP-4	60%	5,686	3,412	2,275
SP-8	50%	3,335	1,668	1,668
QP-1	75%	6,198	4,649	1,550
MC-16	50%	2,856	1,428	1,428

Engineering Calculation Sheet E-8: DNAPL Volume Treated for Development of Alternatve 3

Site: Quendall Terminals

	Engineer	Date
Calculations By:	SDM	8/20/2013
Checked By:	DAH	10/9/2013

Calculations:

Estimate the DNAPL Volume Treated Under Scenarios 1, 2, 3, and 4 to develop Alternative 3

Assumptions:

Equations:

Volume of DNAPL Contaminated Soil = DNAPL Thickness x Area

Volume of Treated DNAPL (gal) = soil volume (ft^3) ÷ 27 ft^3/yd^3 x 1.6 $tons/yd^3$ × 909 kg/ton × 0.011 $kg_{BTEX+PAHs}/kg_{soil}$ × 3.05 $kg_{hydrocarbons}/kg_{BTEX+PAHs}$ × 264 gal/m^3 ÷ 1,040 $kg_{hydrocarbons}/m^3$

Cell	Boring	Total DNAPL Thickness in Feet	Thissen Polygon Area in Square Feet	Total Volume of DNAPL Contaminated Soil Treated in Cu ft.
	Q9	8.5	2,839	
	Q2-C	1.0	1,868	
	Q4	2.5	2,436	
	Q2-D	11.0	1,834	
	BH-30C	3.3	3,558	
1	Total Soil Volume Treated in Cu ft.:			63,828
	HC-5	2.5	5,429	
	MC-20	2.5	9,527	
	MC-23	2.5	6,507	
2			Total Soil in Cu ft.:	53,657
	BH-9	2.0	21,173	
	HC-4	1.0	20,752	
	MC-18	0.8	12,477	
3			Total Soil in Cu ft.:	73,079
	MC-1	8.8	3,840	
4			Total Soil in Cu ft.:	33,603
	SP-7	3.2	9,810	
	BH-23	5.5	9,113	
	SP-6	3.5	9,418	
	SP-5	6.0	7,037	
	RB-19	1.6	7,274	
5			Total Soil in Cu ft.:	168,334
	BH-5B	2.5	3,076	
	BH-5	2.0	5,879	
	RB-12	0.4	4,639	
	SP-3	2.0	5,073	
	SP-4	1.9	3,412	
	QP-5	1.0	4,210	
	SP-8	2.2	1,668	
	BH-20C	1.0	5,542	
6			Total Soil in Cu ft.:	51,352

Scenario	Cell	Soil Volume in Cu. Ft.	DNAPL Volume Treated in Gallons
	1	63,828	
1	Total DNAPL Volume in Gallons:		29,281
	1	63,828	
	2	53,657	
2	Total DNAPL Volume in Gallons:		53,897
	1	63,828	
	2	53,657	
	3	73,079	
3	Total DNAPL Volume in Gallons:		87,422
	1	63,828	
	4	33,603	
	5	168,334	
	6	51,352	
4	Total DNAPL Volume in Gallons:		145,480

Notes:

Total DNAPL thickness and area of Thiessen polygons from Table G-5 of the RI Report

Engineering Calculation Sheet E-9: Removal of DNAPL by Excavation for All Alternatives

			Engineer	Date
Site:	Quendall Terminals			
Calculations:	Estimate the volume of DNAPL to be removed under all alternatives by excavation	Calculations By:	ELG	8/7/2013
		Checked By:	JJP	8/14/2013

Assumptions:

18% of soil removed for PRB, DNAPL Collection Trench, and QP-U DNAPL Upland excavations contain DNAPL

Equations: Volume of Excavated DNAPL (gal) = soil volume (yd^3) x 1.6 $tons/yd^3$ × 909 kg/ton × 0.011 $kg_{BTEX+PAHs}/kg_{soil}$ × 3.05 $kg_{hydrocarbons}/kg_{BTEX+PAHs}$ × 264 gal/m^3 ÷ 1,040 $kg_{hydrocarbons}/m^3$
(see Appendix G of the RI Report)

Volume of DNAPL-containing soil excavated from PRB - see calculation sheet E-2
367 BCY
Volume of DNAPL-containing soil excavated from DNAPL collection trenches - see calculation sheet E-2
167 BCY
Volume of DNAPL excavated in Alternatives 4 and 6 - see calculation sheet E-3
28,315 Gallons
Volume of all DNAPL in upland soil- see calculation sheet E-7
377,462 Gallons
Volume of DNAPL to 15-feet below ground surface in upland soil - see calculation sheet E-7
273,113 Gallons
Volume of DNAPL in T-dock and Nearshore Quendall Pond areas (DA-1, DA-2, and DA-6) - see calculation sheet E-7
60,560 Gallons
Volume of all DNAPL-containing Nearshore/Offshore sediment - see calculation sheet E-7
67,646 Gallons

Alternative	Volume of Excavated Upland DNAPL in Gallons	Volume of Excavated DNAPL from Trenchwork in Gallons	Volume of Excavated Sediment DNAPL in Gallons	Total DNAPL Removed in Gallons
3	--	6,606	--	6,606
4	28,315 (1)	6,606	60,560 (4)	95,481
5	--	4,542	60,560 (4)	65,102
6	28,315 (1)	4,542	60,560 (4)	93,417
7	--	--	67,646 (5)	67,646
8	377,462 (2)	--	67,646 (5)	445,107
9	273,113 (3)	--	67,646 (5)	340,759
10	377,462 (2)	--	67,646 (5)	445,107

Notes:
(1)QP-U DNAPL area only
(2)All upland DNAPL
(3)All upland DNAPL to 15-feet below ground surface
(4)Nearshore Quendall Pond area sediment and T-dock sediment DNAPL
(5)Includes all nearshore and offshore DNAPL

Conversion factors:
1 cy = 27 CF

Engineering Calculation Sheet E-10: Deep Solidification Volumes for Alternatives 3, 5, and 6

			Engineer	Date
Site:	Quendall Terminals			
Calculations:	Estimate the volume in areas of deep DNAPL solidification (RR DNAPL Area & MC-1) for Alternatives 3, 5, and 6	Calculations By:	ELG	8/7/2013
	Estimate the volume of soil to be solidified with 4-ft auger in areas of deep DNAPL solidification	Checked By:	JJP	8/14/2013
	Estimate the volume of soil to be solidified with 8-ft auger in areas of deep DNAPL solidification			

Assumptions:

Solidification of Deeper Alluvium DNAPL requires a 4-foot-diameter auger.

Area of solidification equivalent to area of Thiessen polygons around borings MC-1, BH-30C, Q2-D, Q2-C, Q4, and Q9 (see Table G-5 of the RI Report).

Equations:

Volume = Area x Depth

Volume of Solidified DNAPL (gal) = soil volume (yd^3) x 1.6 tons/yd^3 × 909 kg/ton × 0.011 $kg_{BTEX+PAHs}/kg_{soil}$ × 3.05 $kg_{hydrocarbons}/kg_{BTEX+PAHs}$ × 264 gal/m^3 ÷ 1,040 $kg_{hydrocarbons}/m^3$

(see Calculation-Sheet E-7)

4-ft Diameter Auger

Boring	Area in Square Feet	Maximum Depth in Feet[1]	Volume of Solidified Soil in BCY	Volume of DNAPL-Containing Soil in BCY[2]	Volume of DNAPL Solidified in Gallons
BH-30C	3,558	36	4,711	428	5,305
MC-1	3,840	34	4,765	1,245	15,416
Total:			**9,476**	**1,673**	**20,721**

8-ft Diameter Auger

Boring	Area in Square Feet	Maximum Depth in Feet[1]	Volume of Solidified Soil in BCY	Volume of DNAPL-Containing Soil in BCY[2]	Volume of DNAPL Solidified in Gallons
Q2-D	1,834	32	2,174	747	9,255
Q2-C	1,868	20	1,384	69	857
Q4	2,436	19	1,669	226	2,794
Q9	2,839	27	2,839	894	11,071
Total:			**8,066**	**1,936**	**23,976**

Notes:

8-ft-diameter auger used to solidify shallow soils

4-ft-diameter auger used to solidify deep soils

[1]Maximum depth is maximum depth of DNAPL (see Sheet E-7) in boring plus 2 feet

[2]Volume of DNAPL-containing soil from Sheet E-7

Conversion factors:

1 cy = 27 CF

Engineering Calculation Sheet E-11: QP-U DNAPL Area Solidification - Alternative 5

Site:	Quendall Terminals		**Engineer**	**Date**
Calculations:	Estimate the volume of solidification in the QP-U DNAPL area for Alternative 5	Calculations By:	ELG	8/7/2013
	Estimate the volume of soil to be solidified with 8-ft auger	Checked By:	JJP	8/14/2013
	Estimate the volume of DNAPL solidified			

Assumptions:

18% of solidified soil contains DNAPL[(1)]

Equations: Volume = Area x Depth

8-ft Diameter Auger

Area	Area in Square Feet	Average Solidification Depth in Feet[(2)]	Maximum Solidification Depth in Feet[(3)]	Volume of Solidified Soil in BCY	Volume of DNAPL-Containing Soil in BCY[(4)]	Volume of DNAPL Solidified in Gallons[(4)]
QP-U DNAPL Area	21,556	18	21	14,287	2,284	28,294

Notes:

8-ft-diameter auger used to solidify shallow soils

[(1)]Based on site-wide ratio of DNAPL-containing soil volume to DNAPL-containing soil and overburden soil volume (see Table G-6 of the RI Report).

[(2)]Average depth is average depth of DNAPL in borings (see Sheet E-3) plus 2 feet

[(3)]Maximum depth is maximum depth of DNAPL in borings (see Sheet E-3) plus 2 feet

[(4)]Based on hazardous soil volume - see Calculation Sheet E-3

Conversion factors:

1 cy = 27 CF

Engineering Calculation Sheet E-12: Shallow Solidification Volumes for Alternatives 5 and 6

Site:	Quendall Terminals		**Engineer**	**Date**
Calculations:	Estimate the volume of soil to be solidified in the shallow DNAPL solidification areas for Alternatives 5 and 6	Calculations By:	ELG	8/7/2013
	Estimate the volume of DNAPL in that soil	Checked By:	JJP	8/14/2013

Assumptions: Area of solidification equivalent to area of Thiessen polygons around the borings listed below (see Table E-7).

Equations:
Volume = Area x Depth
Volume of DNAPL Containing Soil = Area x DNAPL Thickness
Volume of Solidified DNAPL (gal) = soil volume (yd^3) x 1.6 tons/yd^3 × 909 kg/ton × 0.011 $kg_{BTEX+PAHs}/kg_{soil}$ × 3.05 $kg_{hydrocarbons}/kg_{BTEX+PAHs}$ × 264 gal/m^3 ÷ 1,040 $kg_{hydrocarbons}/m^3$
(see Sheet E-7)

				Alternative 5 - Thickness > 4 Feet			Alternative 6 - Thickness > 2 Feet		
Boring	Area in Square Feet	Solidification Depth in Feet	DNAPL-Containing Soil Thickness in Feet(1)	Volume of Solidified Soil in BCY(2)	Volume of DNAPL-Containing Soil in BCY	Volume of DNAPL Solidified in Gallons	Volume of Solidified Soil in BCY	Volume of DNAPL-Containing Soil in BCY	Volume of DNAPL Solidified in Gallons
30% of BH-5	1764	20	2.0				1,306	131	1,618
90% of BH-5B	2768	20	2.5				2,051	256	3,175
BH-8	18,456	20	4.0	13,671	2,734	33,868	13,671	2,734	33,868
BH-9	21,173	20	2.0				15,683	1,568	19,426
BH-21A	4,773	20	5.5	3,536	972	12,043	3,536	972	12,043
BH-23	9,113	20	4.5	6,750	1,519	18,813	6,750	1,519	18,813
HC-2	14,230	20	3.9				10,541	2,055	25,460
HC-5	5,429	20	2.5				4,022	503	6,227
HC-7	5,455	20	6.5	4,041	1,313	16,267	4,041	1,313	16,267
MC-7	2,389	20	2.0				1,770	177	2,192
MC-8	1,546	20	3.0				1,145	172	2,128
MC-20	9,527	20	2.5				7,057	882	10,927
MC-23	6,507	20	2.5				4,820	602	7,462
Q1-D	5,496	20	4.0	4,071	814	10,085	4,071	814	10,085
75% of QP-1	4,649	20	2.0				3,443	344	4,265
QP-7	13,112	20	2.3				9,713	1,093	13,534
RB-9	6,694	20	5.0	4,959	1,240	15,355	4,959	1,240	15,355
RB-11	2,810	20	2.0				2,081	208	2,578
RB-23	6,539	20	4.0	4,844	969	11,999	4,844	969	11,999
SP-5	7,037	20	6.0	5,212	1,564	19,369	5,212	1,564	19,369
SP-6	9,418	20	3.5				6,976	1,221	15,121
SP-7	9,810	20	3.2				7,267	1,163	14,402
Total:				**47,084**	**11,125**	**137,800**	**124,959**	**21,501**	**266,315**

Notes:
(1)Thickness of DNAPL containing soil above 20 feet - see calculation sheet E-7
(2)Alternative 5 also includes solidification of QP-U DNAPL Area - see calculation sheet E-11.

Conversion factors:
1 cy = 27 CF

Engineering Calculation Sheet E-13: Solidification Volumes for Alternative 7

Site:	Quendall Terminals		**Engineer**	**Date**
Calculations:	Estimate the volume of upland soil to be solidified with 4-ft auger for Alternative 7	Calculations By:	ELG	8/7/2013
	Estimate the volume of upland soil to be solidified with 8-ft auger for Alternative 7	Checked By:	JJP	8/14/2013

Assumptions:

- 8-ft-diameter auger used to solidify areas where solidification is limited to the Shallow Alluvium.
- 4-ft-diameter auger used to solidify areas including Deeper Alluvium soils.
- Deep DNAPL area includes only BH-30 and MC-1.
- Total volume of solidified soil equal to volume of soil removed under Alternative 8 (see calculation sheet E-4) plus 2 feet below over area of solidification.
- Volume of DNAPL solidified equal to that removed by Alternative 8.
- 4-ft-diameter auger area based on Thiessen polygon area around each boring.

Equations: Volume = Area x Depth

Volume of Soil Excavated under Alternative 8 - see calculation sheet E-4
210,100 BCY

Area of Solidification[1]
420,865 Square Feet

Thickness of solidification below maximum DNAPL extent
2 Feet

Extra Volume of solidified soil
31,175 BCY

Total volume of solidified soil
241,275 BCY

4-ft Diameter Auger

Boring	Area in Square Feet	Maximum Depth in Feet[2]	Volume of Solidified Soil in BCY
BH-30C	3,558	36	4,711
MC-1	3,840	34	4,765
Total:	**7,398**		**9,476**

Total volume to be solidified with 8-foot diameter auger
231,799 BCY

Notes:
Polygon areas for borings BH-30C and MC-1 from RI Table G-5
(1) Area of solidification calculated by AutoCad based on Figure 6-13.
(2) Solidification depth is maximum depth of DNAPL in boring plus 2 feet

Conversion factors:
1 acre = 43,560 SF
1 cy = 27 CF

Engineering Calculation Sheet E-14: Increase in Volume of Soil from Solidification for All Alternatives

Site:	Quendall Terminals		**Engineer**	**Date**
Calculation:	Estimate the volume increase of upland soil during solidification	Calculations By:	ELG	8/7/2013
		Checked By:	JJP	8/14/2013

Assumptions:

20% Increase in soil volume during solidification procedure

Equations: Increase in Volume = Bank Volume x Percentage Increase

Alternative	Volume of Soil to be Solidified in BCY[(1)]	Volume of Solidified Soil in BCY	Increase in Soil Volume in BCY
3	17,542	21,050	3,508
5	78,913	94,695	15,783
6	142,501	171,001	28,500
7	241,275	289,530	48,255
9	362,900	435,480	72,580

Notes:
[(1)]See calculation sheets E-5, E-10, E-11, E-12 and E-13

Engineering Calculation Sheet E-15: Estimated Recovery from DNAPL Collection Trench

Site:	Quendall Terminals		**Engineer**	**Date**
Calculations:	Estimate the volume of DNAPL collected in DNAPL collection trenches	Calculations By:	JJP	5/2/2012
		Checked By:	DAH	6/19/2012

Assumptions:

Initial recovery rate and long-term recovery rate based on pumping test pilot study utilizing 3 wells (see RI Report Figure 4.3-1)

Equations:

Yearly Reduction = (Year 1 Removal Rate - Year 2 Removal Rate) / Year 1 Removal Rate

DNAPL Removal Rate, Full Scale = DNAPL Removal Rate, Pilot Test x $\left(\frac{Full\ Scale\ Effective\ Length\ of\ Trench}{Pilot\ Test\ Effective\ Length\ of\ Influence}\right)$

Pilot Test Removal Rate - Year 1	76	gal/yr
Pilot Test Removal Rate - Year 2	53	gal/yr
Yearly Reduction in Removal Rate	30%	
Pilot Test - Assumed Radius of Influence	10	ft
Pilot Test - Effective LF of Influence	188	lf
Full Scale - Effective LF of Trench	1000	lf

Year	DNAPL Removal Rate in Gallons per Year	Total DNAPL Removed in Gallons
1	403	403
2	282	686
3	198	883
4	138	1022
5	97	1119
6	68	1186
7	47	1234
8	33	1267
9	23	1290
10	16	1307
11	11	1318
12	8	1326
13	6	1332
14	4	1336
15	3	1338
16	2	1340

Notes:
Effective LF of influence is the circumference of the well area of influence at 10-foot radius for 3 wells.
Effective LF of trench assumes both sides of 500-foot-long trench.
LF = liner feet

Engineering Calculation Sheet E-16: Arsenic Treatment Breakthrough Time

Site:	Quendall Terminals		**Engineer**	**Date**
Calculations:	Estimate capacity of arsenic treatment media	Calculations By:	ELG	8/7/2013
	Estimate breakthrough time and lifetime of treatment vessels	Checked By:	JJP	8/14/2013

Equations:

Time to breakthrough = Capacity / Concentration / Pumping Rate

Amount of Arsenic Removed = Dewatering Period x Pumping Rate x Arsenic Concentration

	Parameter	Value		Notes
Arsenic Media				
	Media type	Ferric Adsorptive Media		
	Number of vessels	2	ea	
	Size of vessels	3000	lb	
	Media capacity	4%	by weight	Provided by vendor
	Media capacity	240	lbs arsenic	
	Maximum dewatering period	1,752	days	Alternative 10 - See Section 6.3.10.1.4
	Average Groundwater pumping rate	210	gpm	Alternative 10
	Arsenic concentration	39	ug/L	Average Plume Concentration - See Table A-2
	Time to Breakthrough	2,439	days	
	Amount of arsenic removed	172	lbs	

Notes:

Conversion Factors:
1 Gallon = 3.785 Liters
1 lb = 453,592,000 ug
1 day = 1440 min

Engineering Calculation Sheet E-17: Permeable Treatment Wall GAC Volume and Breakthrough Time

Site:	Quendall Terminals		**Engineer**	**Date**
Calculations:	Estimate volume of granular activated carbon (GAC) for treatment wall	Calculations By:	ELG	8/7/2013
	Estimate breakthrough time and lifetime of treatment wall	Checked By:	JJP	8/14/2013

Assumptions:

- Treatment wall consists of two 100-foot gate sections
- Groundwater velocity based on model (see Appendix A)
- Carbon usage rate of 1.9 lb/1000gal based on vendor modeling
- GAC density is 37.5 lb/ft^3
- Effective lifetime is assumed to be approximately 50% of breakthrough time

Equations:

Carbon Usage Rate (ft$^3_{GAC}$/ft$^3_{Water}$) = Carbon Usage Rate (lb/1000gal) / GAC Density (lb/ft$^3_{GAC}$) x 7.48 (gal/ft$^3_{Water}$)

Time to Breakthrough (years) = Carbon Usage Rate (ft^3/ft^3) / Site Groundwater Flowrate (ft^3/ft^2/day) / Volume per unit area (ft^3/ft^2) / 365 (days/year)

Volume = Depth x Width x Length

Mass = Density x Volume

Parameter		Value	Unit	Notes/Assumptions
Treatment Wall				
	Minimum Width	2	ft	
	Length	200	ft	
	Average Treatment Media Height	22	ft	
Carbon Compositions Calculations				
	Carbon Usage Rate	0.00038	ft3 GAC/ft3 water	based on maximum groundwater concentrations
	Treatment Gate Average Groundwater Velocity	0.90	ft/day	See Appendix A
	Porosity	0.30		
	Treatment Gate Average Groundwater Flowrate	0.27	ft3/ft2/day	
	Wall Width	2	ft	
	GAC Composition	100	percent	
	Volume of GAC in Wall	2.0	ft3/ft2	
	Time to Breakthrough	53.5	years	
	Target Lifetime	22	years	
Earthwork Calculations				
	Average Width	2.0	ft	
	Average Depth	25	ft	
	Volume of Soil Excavated	370	cy	
	Volume GAC	326	cy	
	Volume Structural Fill	44	cy	
	Mass of Soil Excavated	630	tons	Assumed density of 1.7 tons per cubic yard
	Mass GAC	163	tons	Assumed density of 0.5 tons per cubic yard
	Mass Structural Fill	71	tons	Assumed density of 1.6 tons per cubic yard

Conversions:

1 cubic foot = 7.48 gallons

1 year = 365 days

Engineering Calculation Sheet E-18: Average Excavation Depth for Alternatives 4 and 6

			Engineer	**Date**
Site:	Quendall Terminals			
Calculations:		Calculations By:	ELG	8/7/2013
	Estimate the average excavation depth for Alternatives 4 and 6	Checked By:	JJP	8/14/2013

Assumptions:

Equations:

Average = ∑ [(Polygon Area/Total Area) x Maximum DNAPL Depth]

Cell	Boring	Maximum DNAPL Depth in Feet[1]	Thiessen Polygon Area in Feet[1]	Average Depth in Feet
QP-U DNAPL Area	SP-3	16.0	6,341	
	SP-4	12.5	5,686	
	SP-8	18.0	3,335	
	QP-5	12.0	4,210	
	RB-12	18.0	4,639	
	BH-5	19.0	5,879	
		Total Area:	30,090	
			Average Depth in Feet:	15.9

Notes:

[1]Polygon areas and maximum DNAPL depth from RI Table G-5.

Engineering Calculation Sheet E-19: Average Excavation Depths for Each Cell in Alternative 8

Site:	Quendall Terminals		**Engineer**	**Date**
Calculations:		Calculations By:	ELG	8/7/2013
	Estimate the average depth of each excavation cell for Alternative 8	Checked By:	JJP	8/14/2013

Assumptions:

Equations:

Average Depth = ∑ [(Polygon Area/Total Area) x Maximum DNAPL Depth]

Cell	Boring	Maximum DNAPL Depth in Feet	Thissen Polygon Area in Square Feet	Average Excavation Depth in Feet
	BH-30C	33.7	3,558	
	Q2-D	30.0	1,835	
	Q4	16.5	2,437	
	Q2-C	18.0	1,868	
	Q9	25.0	2,839	
		Total Area:	12,537	
1		Average Depth in Feet:		25.5
	Q7	19.0	4,776	
	Q1-D	22.0	5,496	
		Total Area:	10,272	
2		Average Depth in Feet:		20.6
	BH-8	12.5	18,456	
	QP-7	13.8	13,112	
	QP-6	12.3	9,872	
	MC-18	13.0	12,477	
	HC-4	10.0	20,752	
	BH-9	3.5	21,173	
	HC-5	13.0	5,429	
	MC-20	12.3	9,527	
	MC-23	12.8	6,507	
	HC-8	0.0	5,823	
	Q5	0.0	7,255	
	Q6	0.0	6,677	
		Total Area:	137,060	
3		Average Depth in Feet:		9.1
	SP-6	13.0	9,418	
	SP-7	17.8	9,810	
	HC-2	15.0	14,230	
	RB-19	12.6	7,274	
	SP-5	16.0	7,037	
	RB-23	12.0	6,539	
	SWB-4A	11.0	6,404	
	SWB-4	14.0	1,619	
		Total Area:	62,331	
4		Average Depth in Feet:		14.2
	BH-23	24.0	9,113	
		Total Area:	9,113	
5		Average Depth in Feet:		24.0
	BH-20C	26.5	5,542	
		Total Area:	5,542	
6		Average Depth in Feet:		26.5
	SP-8	18.0	3,335	
	QP-5	12.0	4,210	
	BH-5	19.0	5,879	
	BH-5B	16.0	3,076	
	RB-9	20.2	6,694	
	SP-4	12.5	5,686	
	SP-3	16.0	6,341	
	RB-12	18.0	4,639	
	SP-2	22.0	12,915	
	QP-1	18.5	6,198	
	RB-11	18.0	2,810	
	RB-14	21.0	3,800	
	BH-19B	0.0	4,137	
	RB-10	0.0	7,141	
	RB-13	0.0	7,646	
		Total Area:	84,507	
7		Average Depth in Feet:		14.1
	MC-16	13.0	2,856	
	MC-8	14.0	1,546	
	MC-7	18.0	2,389	
	BH-21A	19.0	4,773	
		Total Area:	11,564	
8		Average Depth in Feet:		16.6
	HC-7	15.0	5,455	
	MC-14	0.0	1,983	
	SP-1	9.8	2,699	
	MC-2	14.5	3,755	
		Total Area:	13,892	
9		Average Depth in Feet:		11.7
	MC-1	31.5	3,840	
	MC-13	18.3	4,291	
		Total Area:	8,131	
10		Average Depth in Feet:		24.5

Notes:
Maximum depth and area of Thiessen polygons from Table G-5 of the RI Report

Engineering Calculation Sheet E-20: Dewatering Rate Estimates for Alternative 8

			Engineer	Date
Site:	Quendall Terminals			
Calculations:	Estimate dewatering rate and total amount dewatered for Alternative 8	Calculations By:	ELG	8/7/2013
	Estimate excavation duration	Checked By:	JJP	8/14/2013

Assumptions:

Estimated Deep Aquifer dewatering rates based on modeling for wet excavations - see Appendix A

Assumes dewatering volumes due to internal storage and precipitation are incidental.

Cell	Area in Square Feet	Area in Acres(1)	Maximum Excavation Depth in Feet(2)	Average Exposed Depth in Feet	Estimated Dewatering Rate in GPM	Excavation Duration in Days(3)	Total Groundwater Flow in MG
1	15,672	0.4	33.7	25.5	91	37	5
2	10,105	0.2	22.0	20.6	0	20	0
3	164,325	3.8	13.8	9.1	0	139	0
4	86,433	2.0	17.8	14.2	137	114	22
5	12,616	0.3	24.0	24.0	0	28	0
6	5,773	0.1	26.5	26.5	68	15	1
7	74,327	1.7	22.0	14.1	207	97	29
8	14,529	0.3	19.0	16.6	47	23	2
9	24,276	0.6	15.0	11.7	0	27	0
10	12,809	0.3	31.5	24.5	119	29	5
Totals	**420,865**	**9.7**				**529**	**64**

Notes:

(1)Maximum Excavation Depth based on maximum depth of DNAPL observed within cell area - see calculation sheet E-19

(2)Average Exposed Depth based on average excavation depth of each cell - see calculation sheet E-19

(3)Excavation duration based on 400 cy/day removal/fill rate

Conversion factors:

1 acre = 43,560 SF

Engineering Calculation Sheet E-21: Dewatering Rate Estimates for Alternative 9

Site:	Quendall Terminals		Engineer / Date
Calculation:	Estimate dewatering quantities for Alternative 9	Calculations By:	ELG 8/16/2013
		Checked By:	DAH 10/9/2013

Assumptions:

Dewatering of leakage into excavation cell only: no depressurization of Deep Aquifer

3	feet	yearly precipitation rate	
0.25	feet/day	maximum daily precipitation rate	
4.0	year	Duration of dewatering	Assumes 400 cy/day removal soil rate and 600 cy/day solidification rate
0.3		porosity	
5	feet	depth to water	
15	feet	average depth of excavation	see calculation sheet E-5
56	gpm	leakage rate into excavation cell	
14	acre	total area of excavation	see calculation sheet E-5

Dewatering Flow Rate due to Storage

Storage Volume = Volume of Saturated Soil Removed x Porosity

Volume of Saturated Soil Removed = (Average Depth of Excavation - Average Depth to Water) x Area of Excavation

13,834,350 gallons

Average Storage Flow Rate = Storage Volume x Duration

7 gpm

Dewatering Flow Rate due to Precipitation

Maximum Flow Rate = Maximum Precipitation Rate x Cell Area

Average Flow Rate = Annual Preciptation Rate x Cell Area

Storage Flow Rate = Storage Volume x Duration

	Flow Rate in gpm	
Cell Area in acres	Maximum	Average
4	226	7.4

Total Flow Rate = Storage Flowrate + Precipitation Flowrate + Leakage Flowrate

289	gpm	Maximum Flowrate
70	gpm	Average Flowrate

Conversion factors:

1 acre = 43,560 SF

1 CF = 7.48 gal

1 yr = 525,600 min

Engineering Calculation Sheet E-22: Cost Benefit Analysis of Shoring Cutoff Wall for Alternative 10

Site:	Quendall Terminals	Engineer	Date
Calculation:	Cost benefit analysis to estimate the optimum depth and area of shoring cutoff wall	Calculations By: ELG	8/16/2013
		Checked By: DAH	10/9/2013

NOTE: quantities in this cost-benefit calculation are approximate based on nominal cell areas and depths, and have not been adjusted for specific cells proposed in the alternative

Assumptions:

Shoring walls constructed of temporary sheetpiling with tiebacks

Dimensions:

31	feet	Average depth of excavation	see volume calculation sheet E-6
40	feet	Maximum depth of excavation	see volume calculation sheet E-6
65	feet	Minimum embedment depth	60% embedment - see preliminary shoring design criteria
14	acres	Total area of excavation	

Assume square layout

Unit Costs:

$70	sf	Cost per exposed face of shoring	See Appendix F
$15	sf	Cost for extra embedment	See Appendix F
$83	Mgal /yr	Capital cost of P&T system	systems >20M gal/yr (EPA 2001) - 75% percentile used because both VOCs and SVOCs will require treatment. Adjusted for 10 yrs of inflation at 3%
$9	M gal	O&M cost of P&T system	systems >20M gal/yr (EPA 2001) - 75% percentile used because both VOCs and SVOCs will require treatment. Adjusted for 10 yrs of inflation at 3%
	M gal = 1,000 gallons		

Parameters affecting dewatering treatment rate:

3	feet	yearly precipitation rate	
0.25	feet/day	maximum daily precipitation rate	
4.8	year	Duration of dewatering	Assumes 400 cy/day removal soil rate
0.3		porosity	
5	feet	depth to water	

Estimated steady-state dewatering flowrates in gpm (see Appendix A):

	Maximum Flowrate (at shoreline)			Minimum Flowrate (at railroad)			Average Flowrate (Average of Max and Min)		
	Embedment Depth			Embedment Depth			Embedment Depth		
Cell area in acres	55 Feet	75 Feet	95 Feet	55 Feet	75 Feet	95 Feet	55	75	95
2	940	570	400	740	510	360	840	540	380
1	680	350	210	570	310	200	625	330	205
0.5	400	190	*150*	330	160	100	365	175	125
0.25	210	94	*74*	180	79	52	195	87	63

italics indicates value extrapolated from other runs

Procedure:

Estimate the shoring cost for different excavation cell areas and cutoff wall depths

Estimate the P&T cost for different excavation cell areas and cutoff wall depths

Determine dimensions that result in minimum total cost (shoring + P&T)

Shoring Cost

Equations:

Cell Perimeter = 4 x Square Root (Area) — Assumes square layout

Cell Shoring Area = Cell Perimeter x Average Depth

				65 foot embedment		75 foot embedment			95 foot embedment		
Cell area in acres	Cell side length in feet	Cell perimeter in feet	Number of cells	Exposed area of shoring wall in square feet	Shoring Cost	Extra embedment depth in feet	Extra embedded area in square feet	Extra shoring cost	Extra embedment depth in feet	Extra embedded area in square feet	Extra shoring cost
2	295	1,181	7	258,107	$ 18,067,511	10	83,281	$ 1,249,214	30	249,843	$ 3,747,642
1	209	835	14	365,019	$ 25,551,318	10	117,777	$ 1,766,655	30	353,331	$ 5,299,966
0.5	148	590	28	516,215	$ 36,135,021	10	166,562	$ 2,498,428	30	499,686	$ 7,495,284
0.25	104	417	56	730,038	$ 51,102,637	10	235,554	$ 3,533,311	30	706,662	$ 10,599,932

Total Shoring Cost

	Embedment Depth		
Cell area in acres	55 Feet	75 Feet	95 Feet
2	$ 18,067,511	$ 19,316,725	$ 21,815,152
1	$ 25,551,318	$ 27,317,974	$ 30,851,284
0.5	$ 36,135,021	$ 38,633,449	$ 43,630,305
0.25	$ 51,102,637	$ 54,635,948	$ 61,702,569

Dewatering Flow Rate due to Storage

Storage Volume = Volume of Saturated Soil Removed x Porosity

Volume of Saturated Soil Removed = (Average Depth of Excavation - Average Depth to Water) x Area of Excavation

35,843,726 gallons

Average Storage flow rate = Storage Volume x Duration

14 gpm

Dewatering Flow Rate due to Precipitation

Maximum Flow Rate = Maximum Precipitation Rate x Cell Area

Average Flow Rate = Annual Preciptation Rate x Cell Area

Storage Flow Rate = Storage Volume x Duration

	Flow Rate in gpm	
Cell area in acres	Maximum	Average
2	113	3.7
1	57	1.9
0.5	28	0.9
0.25	14	0.5

Capital Cost (based on maximum flowrate)

		55 Foot Embedment			75 Foot Embedment			95 Foot Embedment		
Cell area in acres	Maximum Dewatering Rate - Storage and Precipitation	Maximum Total Flowrate in gpm	Maximum Dewatering flowrate in 1000 gal/yr	Capital Cost	Maximum Total Flowrate in gpm	Maximum Dewatering flowrate in 1000 gal/yr	Capital Cost	Maximum Total Flowrate in gpm	Maximum Dewatering flowrate in 1000 gal/yr	Capital Cost
2	127	1067	560947	$46,739,642	697	433357	$36,108,526	527	344005	$28,663,466
1	71	751	431709	$35,971,230	421	258261	$21,519,054	281	184677	$15,387,828
0.5	42	442	254809	$21,231,434	232	144433	$12,034,595	192	123409	$10,282,816
0.25	28	238	140079	$11,671,813	122	79110	$6,591,654	102	68708	$5,724,984

O&M Cost (based on average flowrate)

		55 Foot Embedment			75 Foot Embedment			95 Foot Embedment		
Cell area in acres	Average Dewatering Rate (gpm)- Storage and Precipitation	Average Dewatering Flowrate	Total Dewatering flow (1000 gal)	Cost	Average Dewatering Flowrate	Total Dewatering flow (1000 gal)	Cost	Average Dewatering Flowrate	Total Dewatering flow (1000 gal)	Cost
2	18	858	2,223,708	$20,919,343	558	1,461,876	$13,752,474	398	1,055,566	$9,930,143
1	16	641	1,668,283	$15,694,229	346	919,148	$8,646,807	221	601,718	$5,660,612
0.5	15	380	1,003,306	$9,438,514	190	520,812	$4,899,496	140	393,840	$3,705,018
0.25	15	210	569,240	$5,355,073	101	293,710	$2,763,055	78	234,301	$2,204,165

Total Dewatering Cost

	Embedment Depth		
Cell area in acres	55 Feet	75 Feet	95 Feet
2	$ 67,658,985	$ 49,860,999	$ 38,593,609
1	$ 51,665,459	$ 30,165,862	$ 21,048,440
0.5	$ 30,669,948	$ 16,934,091	$ 13,987,834
0.25	$ 17,026,886	$ 9,354,709	$ 7,929,150

Total Shoring + Dewatering Cost

	Embedment Depth		
Cell area in acres	55 Feet	75 Feet	95 Feet
2	$ 85,726,496	$ 69,177,724	$ 60,408,761
1	$ 77,216,778	$ 57,483,836	$ 51,899,725
0.5	$ 66,804,969	$ 55,567,540	$ 57,618,139
0.25	$ 68,129,523	$ 63,990,657	$ 69,631,718

Conversion factors:
1 acre = 43,560 SF
1 CF = 7.48 gal
1 yr = 525,600 min

Engineering Calculation Sheet E-23: Alternative 2 Sediment Capping Volumes

Site:	Quendall Terminals	Date		Engineer	
Calculations:	Estimate of offshore capping volumes for Alternative 2		Calculations By:	A. Skwarski	9/10/2013
			Checked By:	G. Gummadi	9/10/2013

Assumptions:

Area includes: All offshore cap areas.
Excavation is required to maintain current Ordinary High Water Line.
Excavation assumes cap depth at the shoreline and meets existing grade to 75' offshore for length of affected shoreline

The Reactive Capping Material (RCM) is an area calculation.
The sand portion of the RCM is a volume calculation.
Amended sand cap would be installed on the existing grade; no offset dredging assumed

Equations: V=[A x D]+[p x (D x 2D/2)

Note: 2nd term accounts for 2H:1V slopes at edge of cap material after placement

V = volume
A = area
D = depth
p = perimeter

Cell Number	Area (ft^2)	Perimeter (ft)	Offset Excavation (Y or N)	Reactive Cap RCM (ft^2)	Reactive Cap Sand (CY)	ENR (CY)	Engineered Sand Cap (CY)	Amended Sand Cap – Attenuation Layer: Organoclay (CY) (10% by weight)	Amended Sand Cap – Attenuation Layer: Sand (CY) (90% by weight)	Amended Sand Cap – Rest of Cap: Sand (CY)	Erosion Protection Area (ft^2): 5 ft below OLWM (90 ft from shoreline)	Erosion Protection Area (ft^2): between 5 and 15 ft below OLWM (between 90 ft and 220 ft from shoreline)
DA-1	77,392	1,166	--	77,392	1,444	--	--	--	--	--	--	--
DA-2	40,622	814	--	40,622	760	--	--	--	--	--	--	--
DA-3	15,370	497	--	15,370	289	--	--	--	--	--	--	--
DA-4	8,699	373	--	8,699	165	--	--	--	--	--	--	--
DA-5	4,276	261	--	4,276	82	--	--	--	--	--	--	--
DA-6	32,165	1,060	--	--	--	--	--	429	1,954	3,773	24,276	10,067
DA-8, <75-ft of OHWM	26,882	--	Yes	26,882	498	--	--	--	--	--	31,997	21,342
DA-8, >75-ft of OHWM	38,001	1,023	No	38,001	713	--	--	--	--	--		
DA-7, <75-ft of OHWM	3,542	246	Yes	3,542	68	--	--	--	--	--	3,542	--
Sand Cap Area <75-ft of OHWM	38,694	--	Yes	--	--	--	2,150	--	--	--	46,752	98,583
Sand Cap Area >75-ft of OHWM	230,116	3,559	No	--	--	--	13,081	--	--	--		
ENR Area	767,136	4,303	No	--	--	14,246	--				--	--
Subtotals (rounded)				**214,800**	**4,100**	**14,300**	**15,300**			**6,156**	**107,000**	**130,000**

ENR (sand) Thickness	0.5 ft	Area (acres)	17.6
Sand Cap Thickness	1.5 ft	Area (acres)	6.2
RCM Reactive Cap Thick	0.5 ft	Area (acres)	4.9
Amended Sand Cap	4.5 ft	Area (acres)	0.7

Notes:
1: Areas and perimeters calculated by AutoCad - Cell locations are shown on Figure 6-1
2: Offshore sediment is not expected to characterize as hazardous

Conversion factors:
1 acre = 43,560 SF
1 CY = 27 CF

Engineering Calculation Sheet E-24: Alternative 3 Sediment Capping Volumes

Site:	Quendall Terminals		Engineer	Date
Calculations:	Estimate of offshore capping volumes for Alternative 3	Calculations By:	G. Gummadi	8/13/2013
		Checked By:	A. Skwarski	8/14/2013

Assumptions:

Area includes: All offshore cap areas.
Excavation is required to maintain current Ordinary High Water Line.
Excavation assumes cap depth at the shoreline and meets existing grade to 75' offshore for length of affected shoreline
The Reactive Capping Material (RCM) is an area calculation.
The sand portion of the RCM is a volume calculation.

Equations: V=[A x D]+[p x (D x 2D/2)]

Note: 2nd term accounts for 2H:1V slopes at edge of cap material after placement

V = volume
A = area
D = depth
p = perimeter

Cell Number	Area (ft^2)	Perimeter (ft)	Offset Excavation (Y or N)	Reactive Cap RCM (ft^2)	Reactive Cap Sand (CY)	ENR (CY)	Engineered Sand Cap (CY)	Erosion Protection Area (ft^2) 5 ft below OLWM (90 ft from shoreline)	Erosion Protection Area (ft^2) between 5 and 15 ft below OLWM (between 90 ft and 220 ft from shoreline)
DA-1	77,392	1,166	--	77,392	1,444	--	--	--	--
DA-2	40,622	814	--	40,622	760	--	--	--	--
DA-3	15,370	497	--	15,370	289	--	--	--	--
DA-4	8,699	373	--	8,699	165	--	--	--	--
DA-5	4,276	261	--	4,276	82	--	--	--	--
DA-6/8, <75-ft of OHWM	47,236	--	Yes	47,236	875	--	--	56,609	40,076
DA-6/8, >75-ft of OHWM	49,813	1,613	No	49,813	937	--	--		
DA-7, <75-ft of OHWM	3,542	246	Yes	3,542	68	--	--	3,542	--
Sand Cap Area <75-ft of OHWM	38,694	--	Yes	--	--	--	2,150	46,752	98,583
Sand Cap Area >75-ft of OHWM	230,116	3,559	No	--	--	--	13,081		
ENR Area	767,136	4,303	No	--	--	14,246	--	--	--
Subtotals (rounded)				**247,000**	**4,700**	**14,300**	**15,300**	**107,000**	**138,700**

ENR (sand) Thickness	0.5 ft	Area (acres)	17.6
Sand Cap Thickness	1.5 ft	Area (acres)	6.2
Reactive Cap Thickness	0.5 ft	Area (acres)	5.7

Notes:
1: Areas and perimeters calculated by AutoCad - Cell locations are shown on Figure 6-4.
2: Offshore sediment is not expected to characterize as hazardous.

Conversion factors:
1 acre = 43,560 SF
1 CY = 27 CF

Engineering Calculation Sheet E-25: Alternative 4, 5 and 6 Sediment Capping Volumes

Site:	Quendall Terminals	Engineer	Date
Calculations:	Estimate of offshore capping volumes for Alternative 4, 5, and 6	Calculations By: G. Gummadi	8/13/2013
		Checked By: A. Skwarski	8/14/2013

Assumptions:

Area includes: All offshore cap areas.
Excavation is required to maintain current Ordinary High Water Line.
Excavation assumes cap depth at the shoreline and meets existing grade 75' offshore for length of affected shoreline
The Reactive Capping Material (RCM) is an area calculation.
The sand portion of the RCM is a volume calculation.

Equations: V=[A x D]+[p x (D x 2D/2)]

Note: 2nd term accounts for 2H:1V slopes at edge of cap material after placement

V = volume
A = area
D = depth
p = perimeter

Cell Number	Area (ft^2)	Perimeter (ft)	Offset Excavation (Y or N)	Reactive Cap: RCM (ft^2)	Reactive Cap: Sand (CY)	ENR (CY)	Engineered Sand Cap (CY)	Erosion Protection Area (ft^2): 5 ft below OLWM (90 ft from shoreline)	Erosion Protection Area (ft^2): between 5 and 15 ft below OLWM (between 90 ft and 220 ft from shoreline)
DA-1	77,392	1,166	--	--	--	--	--	--	--
DA-2	40,622	814	--	--	--	--	--	--	--
DA-3	15,370	497	No	15,370	289	--	--	--	--
DA-4	8,699	373	No	8,699	165	--	--	--	--
DA-5	4,276	261	No	4,276	82	--	--	--	--
DA-6	32,165	1,060	--	--	--	--	--	24,276	10,067
DA-7, <75-ft of OHWM	3,542	246	Yes	3,542	68	--	--	3,542	--
DA-8, <75-ft of OHWM	26,884	--	Yes	26,884	498	--	--	31,997	21,342
DA-8, >75-ft of OHWM	26,820	1,023	No	26,820	506	--	--		
Sand Cap Area <75-ft of OHWM	38,697	--	Yes	--	--	--	2,150	47,095	107,664
Sand Cap Area >75-ft of OHWM	239,204	3,567	No	--	--	--	13,586		
ENR Area	767,136	4,303	No	--	--	14,246	--	--	--
Subtotals (rounded)				**85,600**	**1,700**	**14,300**	**15,800**	**107,000**	**139,100**

ENR (sand) Thickness	0.5 ft	Area (acres)	17.6
Sand Cap Thickness	1.5 ft	Area (acres)	6.4
Reactive Cap Thickness	0.5 ft	Area (acres)	2.0

Notes:
1: Areas and perimeters calculated by AutoCad - Cell locations are shown on Figure 6-7.
2: Offshore sediment is not expected to characterize as hazardous.

Conversion factors:
1 acre = 43,560 SF
1 CY = 27 CF

Engineering Calculation Sheet E-26: Alternative 7 and 8 Sediment Capping Volumes

Site:	Quendall Terminals		Engineer	Date
Calculations:	Estimate of offshore capping volumes for Alternative 7 and 8	Calculations By:	G. Gummadi	8/13/2013
		Checked By:	A. Skwarski	8/14/2013

Assumptions:

Area includes: All offshore cap areas.
Excavation is required to maintain current Ordinary High Water Line.
Excavation assumes cap depth at the shoreline and meets existing grade 75' offshore for length of affected shoreline
In the nearshore, sediment will be offset/dredged at elevations above 11 ft
The Reactive Capping Material (RCM) is an area calculation.
The sand portion of the RCM is a volume calculation.

Equations: V=[A x D]+[p x (D x 2D/2)

Note: 2nd term accounts for 2H:1V slopes at edge of cap material after placement

V = volume
A = area
D = depth
p = perimeter

Cell Number	Area (ft^2)	Perimeter (Feet)	Offset Excavation (Y or N)	ENR (CY)	Engineered Sand Cap (CY)	Erosion Protection Area (ft^2): 5 ft below OLWM (90 ft from shoreline)	Erosion Protection Area (ft^2): Between 5 and 15 ft below OLWM (between 90 ft and 220 ft from shoreline)
DA-1	77,392	1,166	--	--	--	--	--
DA-2	40,622	814	--	--	--	--	--
DA-3	15,370	497	--	--	--	--	--
DA-4	8,699	373	--	--	--	--	--
DA-5/6/8	131,005	1,794	--	--	--	62,117	67,298
DA-7	3,542	246	--	--	--	3,542	--
Sand Cap Area <75-ft of OHWM	34,115	--	Yes	--	1,895	41,197	75,296
Sand Cap Area >75-ft of OHWM	204,948	3,383	No	--	11,668		
ENR Area	767,136	4,303	No	14,246	--	--	--
Subtotals (rounded)				**14,300**	**13,600**	**106,900**	**142,600**

ENR (sand) Thickness	0.5 ft	Area (acres)	17.6
Sand Cap Thickness	1.5 ft	Area (acres)	5.5

Notes:
1: Areas and perimeters calculated by AutoCad - Cell locations are shown on Figure 6-13
2: Offshore sediment is not expected to characterize as hazardous.

Conversion factors:
1 acre = 43,560 SF
1 CY = 27 CF

Engineering Calculation Sheet E-27: Alternative 9 and 10 Sediment Capping Volumes

Site:	Quendall Terminals	Engineer	Date
Calculations:	Estimate of offshore capping volumes for Alternative 9 and 10	Calculations By: G. Gummadi	8/13/2013
		Checked By: A. Skwarski	8/14/2013

Assumptions:

Area includes: All offshore cap areas.
Excavation is required to maintain current Ordinary High Water Line.
Excavation assumes cap depth at the shoreline and meets existing grade 75' offshore for length of affected shoreline
In the nearshore, sediment will be offset/dredged at elevations above 11 ft.
The Reactive Capping Material (RCM) is an area calculation.
The sand portion of the RCM is a volume calculation.

Equations: V=[A x D]+[p x (D x 2D/2)

Note: 2nd term accounts for 2H:1V slopes at edge of cap material after placement

V = volume
A = area
D = depth
p = perimeter

Cell Number	Area (ft^2)	Perimeter (Feet)	Offset Excavation (Y or N)	ENR (CY)	Engineered Sand Cap (CY)	Erosion Protection Area (ft^2): 5 ft below OLWM (90 ft from shoreline)	Erosion Protection Area (ft^2): between 5 and 15 ft below OLWM (between 90 ft and 220 ft from shoreline)
DA-1	77,392	1,166	--	--	--	--	--
DA-2	40,622	814	--	--	--	--	--
DA-3	15,370	497	--	--	--	--	--
DA-4	8,699	373	--	--	--	--	--
DA-7	3,542	246	--	--	--	3,542	--
NA-1, NA-2, NA-3, NA-4, NA-5	200,902	--	--	--	--	85,961	107,012
Sand Cap Area <75-ft of OHWM	14,137	--	Yes	--	785	17,194	38,322
Sand Cap Area >75-ft of OHWM	154,500	2,821	No	--	8,818		
ENR Area	767,136	4,303	No	14,246	--	--	--
Subtotals (rounded)				**14,300**	**9,700**	**106,700**	**145,400**

ENR (sand) Thickness	0.5 ft	Area (acres)	17.6
Sand Cap Thickness	1.5 ft	Area (acres)	3.9

Notes:
1: Areas and perimeters calculated by AutoCad - Cell locations are shown on Figure 6-19.
2: Offshore sediment is not expected to characterize as hazardous.

Conversion factors:
1 acre = 43,560 SF
1 CY = 27 CF

Engineering Calculation Sheet E-28: Alternative 2 Dredging Volumes

Site:	Quendall Terminals		**Engineer**	**Date**
Calculations:	Estimate the volume of sediment to be dredged or excavated for Alternatives 2.	Calculations By:	A. Skwarski	9/10/2013
		Checked By:	G. Gummadi	9/10/2013

Assumptions: Sediment in the nearshore capping areas would be removed to offset for cap and erosion protection. This would include all sediment area above 11 ft elevation.

Equations:
Sediment Removal Volume = A x D
Side-Slope Sediment Removal Volume = P x D x 2D / 2

A = area
D = total depth
P = perimeter

	Mechanical Dredge to Off-Set Capping							
Dredge Area	**Area (ft^2)**	**Perimeter (ft)**	**Target Depth (ft bss)**	**Total Depth (ft bss)**	**Sediment Removal Volume (CY)**	**Side-Slope Sediment Removal Volume (CY)**	**Total Sediment Volume (CY)**	**Target Depth based on**
DA-8, <75-ft of OHWM	26,882	1,613	0.5	0.5	498	14.9	513	Reactive Cap Thickness
DA-7, <75-ft of OHWM	3,542	245.57	0.5	0.5	66	2.3	68	Reactive Cap Thickness
Sand Cap Area < 75-ft of OHWM	38,694	--	1.5	1.5	2,150	--	2,150	Sand Cap Thickness
Subtotal (rounded)							**2,800**	

	Dredging Depth for Offsetting
Reactive Cap Areas	0.5 ft
Sand Cap Area	1.5 ft

Notes:
1. Dredge areas and perimeters calculated by AutoCad - Cell locations are shown on Figure 6-1.
2. Volume estimate is based on plume footprint and 2:1 sideslopes.

Conversion factors:
1 acre = 43,560 SF
1 cy = 27 CF

Engineering Calculation Sheet E-29: Alternative 3 Dredging Volumes

Site:	Quendall Terminals	**Engineer**	**Date**
Calculations:	Estimate the volume of sediment to be dredged or excavated for Alternatives 3.	Calculations By: G. Gummadi	8/13/2013
		Checked By: A. Skwarski	8/14/2013

Assumptions: Sediment in the nearshore capping areas would be removed to offset for cap and erosion protection. This would include all sediment area above 11 ft elevation.

Equations:
Sediment Removal Volume = A x D
Side-Slope Sediment Removal Volume = P x D x 2D / 2

A = area
D = total depth
P = perimeter

	Mechanical Dredge to Off-Set Capping							
Dredge Area	**Area (ft^2)**	**Perimeter (ft)**	**Target Depth (ft bss)**	**Total Depth (ft bss)**	**Sediment Removal Volume (CY)**	**Side-Slope Sediment Removal Volume (CY)**	**Total Sediment Volume (CY)**	**Target Depth based on**
DA-6/8, <75-ft of OHWM	47,236	1,613	0.5	0.5	875	14.9	890	Reactive Cap Thickness
DA-7, <75-ft of OHWM	3,542	245.57	0.5	0.5	66	2.3	68	Reactive Cap Thickness
Sand Cap Area < 75-ft of OHWM	38,694	--	1.5	1.5	2,150	--	2,150	Sand Cap Thickness
Subtotal (rounded)							**3,200**	

	Dredging Depth for Offsetting
Reactive Cap Areas	0.5 ft
Sand Cap Area	1.5 ft

Notes:
1. Dredge areas and perimeters calculated by AutoCad - Cell locations are shown on Figure 6-4.
2. Volume estimate is based on plume footprint and 2:1 sideslopes.

Conversion factors:
1 acre = 43,560 SF
1 cy = 27 CF

Engineering Calculation Sheet E-30: Alternative 4, 5 and 6 Dredging Volumes

Site:	Quendall Terminals		**Engineer** **Date**
Calculations:	Estimate the volume of sediment to be dredged or excavated for Alternatives 4, 5, and 6.	Calculations By:	G. Gummadi 8/13/2013
		Checked By:	A. Skwarski 8/14/2013

Assumptions: Sediment in the nearshore capping areas would be removed to offset for cap and erosion protection. This would include all sediment area above 11 ft elevation.

Equations:
Sediment Removal Volume = A x D
Side-Slope Sediment Removal Volume = P x D x 2D / 2

A = area
D = total depth
P = perimeter

Dredge Area	Area (ft^2)	Perimeter (ft)	Target Depth (ft bss)	Total Depth (ft bss)	Sediment Removal Volume (CY)	Side-Slope Sediment Removal Volume (CY)	Total Sediment Volume (CY)	Reactive Residual Cover: Organoclay (CY) (10% by weight)	Reactive Residual Cover: Sand (CY) (90% by weight)	Backfill (CY)	Core that Target Depth Based on
Hydraulic Dredging (off-shore)											
DA-1	77,392	1,166	2.4	3.4	9,746	97	9,843	260	1,184	8,399	3.8 ft for DNAPL area (half of dredge area) based on VT-4; 1.0 ft of removal within rest of DA-1
DA-2	40,622	814	0.5	1.5	2,257	68	2,325	137	623	1,565	TD-08
DA-3	--	--	--	--	--	--	--	--	--		VS27
DA-4	--	--	--	--	--	--	--	--	--		TD-01
DA-5	--	--	--	--	--	--	--	--	--		EPA-8
Subtotal (Rounded)	**118,100**	**2.7**	**acres**				**12,200**	**400**	**1,900**	**10,000**	
Mechanical Dredging (within sheetpile)											
DA-6	32,165	--	8.2	9.2	10,960	--	10,960	107	488	10,364	Average of VS-30 and QPN-02
Subtotal (Rounded)	**32,200**	**0.7**	**acres**				**11,000**	**110**	**490**	**10,370**	
Mechanical Dredging for Cap Off-Set											
DA-7	3,542	246	0.5	0.5	66	--	66	--	--		Reactive Cap Thickness
DA-8 <75-ft of OHWM	26,884	1,023	0.5	0.5	498	--	498	--	--		Reactive Cap Thickness
Sand Cap Area < 75-ft of OHWM	38,697	--	1.5	1.5	2,150	--	2,150	--	--		Sand Cap Thickness
Subtotal (Rounded)	**69,200**	**1.6**	**acres**				**2,720**				
Total (Rounded)							**25,900**	**510**	**2,300**	**20,400**	

	Dredging Depth for Offsetting
Reactive Cap Areas	0.5 ft
Sand Cap Area	1.5 ft

Notes:
1. Dredge areas and perimeters calculated by AutoCad - Cell locations are shown on Figure 6-7.
2. Total depth assumes nearest observation of NAPL in a boring, and includes 1-foot of overdredge.
3. Offshore sediment is not expected to characterize as hazardous.
4. Volume estimate is based on plume footprint and 2:1 sideslopes.
5. Assumed bulk densities for OC to be 53 lb/ft^3 and sand to be 105 lb/ft^3. This translates to 18% OC and 82% Sand by volume.

Conversion factors:
1 acre = 43,560 SF
1 cy = 27 CF

Engineering Calculation Sheet E-31: Alternative 7 and 8 Dredging Volumes

Site:	Quendall Terminals	**Engineer**	**Date**
Calculations:	Estimate the volume of sediment to be dredged or excavated for Alternatives 7 and 8.	Calculations By: G. Gummadi	8/13/2013
		Checked By: A. Skwarski	8/14/2013

Assumptions: Sediment in the nearshore capping areas would be removed to offset for cap and erosion protection. This would include all sediment area above 11 ft elevation.

Equations:
Sediment Removal Volume = A x D
Side-Slope Sediment Removal Volume = P x D x 2D / 2

A = area
D = total depth
P = perimeter

Dredge Area	Area (ft^2)	Perimeter (ft)	Target Depth (ft bss)	Total Depth (ft bss)	Sediment Removal Volume (CY)	Side-Slope Sediment Removal Volume (CY)	Total Sediment Volume (CY)	Reactive Residual Cover: Organoclay (CY) (10% by weight)	Reactive Residual Cover: Sand (CY) (90% by weight)	Backfill (CY)	Core that Target Depth Based on
Hydraulic Dredging (off-shore)											
DA-1	77,392	1,166	2.4	3.4	9,746	97	9,843	260	1,184	8,399	3.8 ft for DNAPL area (half of dredge area) based on VT-4; 1.0 ft of removal within rest of DA-1
DA-2	40,622	814	0.5	1.5	2,257	68	2,325	137	623	1,565	TD-08
DA-3	15,370	497	2.7	3.7	2,106	252	2,358	52	237	2,069	VS27
DA-4	8,699	373	0.8	1.8	580	45	625	30	135	460	TD-01
Subtotal (Rounded)	**142,100**	**3.3**	**acres**				**15,200**	**480**	**2,200**	**12,500**	
Mechanical Dredging (within sheetpile)											
DA-5	4,276	--	3.0	4.0	634	--	634	14	65	554	EPA-8
DA-6	32,165	--	8.2	9.2	10,960	--	10,960	107	488	10,364	Average of VS-30 and QPN-02
DA-8	53,704	--	11.4	12.4	24,664	--	24,664	179	816	23,669	Average of VS-2, QPN-07 and NS15-C1
Additional Area within sheetpile/slopes	40,860	--	--	2.0	3,027	--	3,027	136	620	2,270	NA
DA-7	3,542	246	13.0	14.0	1,837	--	1,837	12	54	1,771	MC-16
Subtotal (Rounded)	**134,600**	**3.1**	**acres**				**41,200**	**450**	**2,100**	**38,600**	
Mechanical Dredging for Cap Off-Set											
Sand Cap Area < 75-ft of OHWM	34,115	--	1.5	1.5	1,895	--	1,895	--	--		Sand Cap Thickness
Subtotal (Rounded)	**34,200**	**0.8**	**acres**				**1,900**				
Total (Rounded)							**58,300**	**930**	**4,300**	**51,200**	

	Dredging Depth for Offsetting
Sand Cap Area	1.5 ft

Notes:
1. Dredge areas and perimeters calculated by AutoCad - Cell locations are shown on Figure 6-13.
2. Total depth assumes nearest observation of NAPL in a boring, and includes 1-foot of overdredge.
3. Offshore sediment is not expected to characterize as hazardous.
4. Volume estimate is based on plume footprint and 2:1 sideslopes.
5. Assumed bulk densities for OC to be 53 lb/ft^3 and sand to be 105 lb/ft^3. This translates to 18% OC and 82% Sand by volume.

Conversion factors:
1 acre = 43,560 SF
1 cy = 27 CF

Engineering Calculation Sheet E-32: Alternative 9 and 10 Dredging Volumes - Part 1

Site:	Quendall Terminals		**Engineer**	**Date**
Calculations:	Estimate the volume of sediment outside temporary sheetpile wall enclosure to be dredged or excavated for Alternatives 9 and 10.	Calculations By:	G. Gummadi	8/13/2013
		Checked By:	A. Skwarski	8/14/2013

Assumptions: Sediment in the nearshore capping areas would be removed to offset for cap and erosion protection. This would include all sediment area above 11 ft elevation.

Equations:
Sediment Removal Volume = A x D
Side-Slope Sediment Removal Volume = P x D x 2D / 2

A = area
D = total depth
P = perimeter

Dredge Area	Area (ft^2)	Perimeter (ft)	Target Depth (ft bss)	Total Depth (ft bss)	Sediment Removal Volume (CY)	Side-Slope Sediment Removal Volume (CY)	Total Sediment Volume (CY)	Reactive Residual Cover		Backfill (CY)	Core that Target Depth Based on
								Organoclay (CY) (10% by weight)	Sand (CY) (90% by weight)		
Hydraulic Dredging (off-shore)											
DA-1	77,392	1,166	3.4	4.4	12,612	97	12,709	260	1,184	11,265	NAPL Depth plus 2' 5.8 ft for DNAPL area (half of dredge area) based on VT-4; 1.0 ft of removal within rest of DA-1
DA-2	40,622	814	2.5	3.5	5,266	369	5,635	137	623	4,875	NAPL depth plus 2'
DA-3	15,370	497	4.7	5.7	3,245	598	3,843	52	237	3,553	NAPL depth plus 2'
DA-4	8,699	373	2.8	3.8	1,224	200	1,424	30	135	1,259	NAPL depth plus 2'
Subtotal (Rounded)	**142,100**	**3.3**	**acres**				**23,700**	**480**	**2,200**	**21,000**	
Mechanical Dredging for Cap Off-Set											
Sand Cap Area < 75-ft of OHWM	14,137	--	1.5	1.5	785	--	785	--	--	--	Sand Cap Thickness
Subtotal (Rounded)	**14,200**	**0.3**	**acres**				**800**	--	--	--	

	Dredging Depth for Offsetting
Sand Cap Area	1.5 ft

Notes:
1. Dredge areas and perimeters calculated by AutoCad - Cell locations are shown on Figure 6-19.
2. Total depth assumes nearest observation of NAPL in a boring, and includes 1-foot of overdredge.
3. Offshore sediment is not expected to characterize as hazardous.
4. Volume estimate is based on plume footprint and 2:1 sideslopes. This translates to 18% OC and 82% Sand by volume.

Conversion factors:
1 acre = 43,560 SF
1 cy = 27 CF

Engineering Calculation Sheet E-33: Alternative 9 and 10 Dredging Volumes - Part 2

Site: Quendall Terminals

Calculations: Estimate the volume of nearshore sediment to be excavated for Alternatives 9 and 10

	Engineer	Date
Calculations By:	G. Gummadi	8/13/2013
Checked By:	A. Skwarski	8/14/2013

Assumptions:

Area includes: Shallow Alluvium within benzo[a]pyrene and arsenic plumes and includes Nearshore NAPL deposits

Equations:

Sediment Removal Volume = A x D
Side-Slope Sediment Removal Volume = P x D x 2D / 2

A = area
D = total depth
P = perimeter

Cell Number	Total Cell Area (ft^2)	Length of Transect with adjacent dredge cell (ft)	Extent of B[a]P Plume (ft^2)	Target Depth (ft bss)	Total Depth (ft bss)	Perimeter (ft)	Sediment Removal Volume (CY)	Sideslope Sediment Removal Volume (CY)	Total Sediment Volume in CY	Reactive Residual Cover: Organoclay (CY) (10% by weight)	Reactive Residual Cover: Sand (CY) (90% by weight)	Backfill (CY)
Mechanical Dredging (within sheetpile)												
NA-1	65,305	--	3,490	15	15	1,003	36,280	--	36,280	218	992	35,071
NA-2	25,649	192	4,766	27	27	684	25,649	1,025	26,674	85	389	26,199
NA-3	47,961	200	8,482	19	19	872	33,750	473	34,224	160	728	33,336
NA-4	16,680	242	15,015	20	20	619	12,355	8.9	12,364	56	253	12,055
NA-5	45,307	240	12,422	22	22	891	36,917	35	36,952	151	688	36,113
DA-7	3,542	--	2,949	15	16	246	2,099	--	2,099	12	54	2,033
Subtotal (Rounded)	**204,500**	**4.7**	**acres**						**148,600**	**690**	**3,200**	**144,900**

Notes:
1. Dredge areas and perimeters calculated by AutoCad - Cell locations are shown on Figure 6-21.
2. Total depth assumes average depth of B[a]P and arsenic contamination from Section E-E'.
3. Volume estimate is based on plume footprint and 2:1 sideslopes.
4. Approximate dredge elevations in each dredge unit are: NA-1 = -3.5 ft, NA-2 = -12 ft, NA-1 = -6.5 ft, NA-2 = -8 ft, and NA-1 = -11 ft.
5. Assumed bulk densities for OC to be 53 lb/ft^3 and sand to be 105 lb/ft^3. This translates to 18% OC and 82% Sand by volume.

Conversion factors:
1 acre = 43,560 SF
1 cy = 27 CF

Engineering Calculation Sheet E-34: Offshore Duration Estimates of Alternatives

Site: Quendall Terminals
Calculations: Estimate the sediment remedy implementation durations of alternatives

	Engineer	Date
Calculations By:	A. Skwarski	9/10/2013
Checked By:	G. Gummadi	9/10/2013

Assumptions: Rate of implementation of various technologies is based on previous project experience.

		Alternative 2	Alternative 3	Alternatives 4, 5, & 6	Alternatives 7 & 8	Alternatives 9 & 10
Enhanced Natural Recovery	Volume of material (CY)	14,300	14,300	14,300	14,300	14,300
	Rate of material placement (CY/day)	500	500	500	500	500
	Numer of days for implementation (days)	29	29	29	29	29
	Number of weeks for implementation	5	5	5	5	5
Engineered Sand Cap	Volume of material (CY)	15,300	15,300	15,800	13,600	9,700
	Rate of material placement (CY/day)	500	500	500	500	500
	Numer of days for implementation (days)	31	31	32	28	20
	Number of weeks for implementation	5	5	6	5	4
Amended Sand Cap	Volume of material (CY)	6,156	--	--	--	--
	Rate of material placement (CY/day)	500	--	--	--	--
	Numer of days for implementation (days)	13	--	--	--	--
	Number of weeks for implementation	3	--	--	--	--
Reactive Cap	Area to be capped (ft^2)	214,800	247,000	85,600	--	--
	Rate of material placement (ft^2/day)	10,000	10,000	10,000	--	--
	Numer of days for implementation (days)	22	25	9	--	--
	Number of weeks for implementation	4	5	2	--	--
Dredging for Remedy Offsetting	Volume of material (CY)	2,800	3,200	2,720	1,900	800
	Rate of dredging (CY/day)	400	400	400	400	400
	Numer of days for implementation (days)	7	8	7	5	2
	Number of weeks for implementation	2	2	2	1	1
Offshore Hydraulic Dredging	Volume of material (CY)	--	--	12,200	15,200	23,700
	Rate of dredging (CY/day)	--	--	400	400	400
	Numer of days for implementation (days)	--	--	31	38	60
	Number of weeks for implementation	--	--	6	7	10
Sheet Pile Containment - Installation	Total length (linear ft)	--	--	700	1,260	1,531
	Rate of installation (linear ft/day)	--	--	20	20	20
	Number of days for installation (days)	--	--	36	64	77
	Number of weeks for implementation	--	--	6	11	13
Nearshore Mechanical Dredging	Volume of material (CY)	--	--	11,000	41,200	148,600
	Rate of dredging (CY/day)	--	--	400	400	400
	Numer of days for implementation (days)	--	--	28	103	372
	Number of weeks for implementation	--	--	5	18	62
Sheet Pile Containment - Removal	Total length (linear ft)	--	--	700	1,260	1,531
	Rate of removal (linear ft/day)	--	--	30	30	30
	Number of days for removal (days)	--	--	24	43	52
	Number of weeks for implementation	--	--	4	8	9
Residual Reactive Cover	Volume of material (CY)	--	--	2,810	5,230	6,570
	Rate of material placement (CY/day)	--	--	500	500	500
	Numer of days for implementation (days)	--	--	6	11	14
	Number of weeks for implementation	--	--	1	2	3
Backfilling	Volume of material (CY)	--	--	20,400	51,200	165,900
	Rate of material placement (CY/day)	--	--	500	500	500
	Numer of days for implementation (days)	--	--	41	103	332
	Number of weeks for implementation	--	--	7	18	56
	Total Duration (weeks)	**16**	**17**	**44**	**75**	**163**

Notes:
1. Volumes of materials is estimated from dredge areas and perimeters estimated in AutoCad.
2. One week assumes 6 work days per week. All weeks have been rounded up to the nearest whole week.

APPENDIX F

Construction Shoring Design Considerations

Project No.: 020027-010

June 14, 2012

To: Jeremy Porter
Aspect Consulting LLC

From: **Andrew J. Holmson, EIT**
Project Geotechnical Engineer

John L. Peterson, PE
Senior Associate Geotechnical Engineer

Re: **Excavation and Shoring Considerations – Quendall Terminals**

This memorandum summarizes the preliminary excavation and shoring considerations of Aspect Consulting, LLC (Aspect) for the proposed excavation alternatives being considered for incorporation into the environmental remediation project at the Quendall Terminals property located in Renton, Washington (Site).

Current environmental remediation plans include alternatives for excavation and removal of contaminated soil within the property. Multiple excavation scenarios are being considered. In general the two types of scenarios consist of:

1. Fully dewatered (dry) excavation scenarios that would include excavation and removal of contaminated soils to depths ranging from 20 to 40 feet below the existing Site grades over areas ranging from 1 to 2 acres.

2. Partially dewatered (wet) excavation scenarios that would include excavation and removal of contaminated soils to depths ranging from 10 to 34 feet below the existing Site grades.

Variations of the two scenario types described above are also being considered including breaking the proposed larger excavations areas into smaller, segmented cell excavations. Shallow groundwater conditions across the Site will require dewatering and/or impermeable shoring as part of the excavation and removal processes.

Site Geology

Generally, the Site geology within the depth range of the proposed excavation alternatives can be broken into three separate units for geotechnical engineering purposes.

Our characterization of subsurface conditions suggests the Site is underlain by a surface layer of Fill that is variable in composition and density, and is generally on the order of 8 to 10 feet thick. The Fill mantles a sequence of very soft Shallow Alluvium ranging in thickness from about 20 to 35 feet and consisting primarily of fine-grained organic-rich and peaty soils with scattered loose sand layers. The Deeper Alluvium consists of generally more competent sands and gravels to a depth of 130 feet or more. Very soft, fine-grained Lacustrine deposits were encountered beneath the alluvium. Competent, glacially consolidated soil and/or bedrock were encountered beneath the alluvium on the adjacent shoreline properties (Football Northwest to the north), but were not encountered in explorations on the Site.

Groundwater

Over twenty groundwater monitoring wells are located on the project Site. Groundwater is typically encountered between approximately 2 and 10 feet below the existing Site grades, with groundwater flow generally east to west/northwest direction toward the lake. Vertical groundwater flow gradients in the Shallow and Deeper Alluvium units at the Site exhibit downward gradients along the eastern portion of the Site becoming upward near the lake shoreline.

The shallow groundwater across the Site would present construction challenges for trenching and excavating below the water table. Construction dewatering should be anticipated for these deep excavations. If deep excavations occur after parts of the Site are developed, construction dewatering plans will have to consider the potential of dewatering-induced settlement caused by drawdown of the water table. Any dewatering activities will need to consider health, safety, and water treatment issues associated with potential exposure to and extraction of dissolved phase chemical constituents in groundwater.

Excavation and Shoring Considerations

Shoring Alternatives

Taking into consideration the Site geology, groundwater conditions, and proposed excavation and removal alternatives, steel sheet piles are likely the most practical and cost-effective method for support of the large excavations being considered.

Steel sheet piling can be installed and configured to achieve an impermeable shoring system to help reduce the amount of dewatering required for the proposed excavations. Sheet piling can also be salvaged and possibly re-used in a scenario involving a segmented approach of multiple, smaller excavation cells. Sheet piles can be installed as a cantilever system to support an excavation height of approximately 16 feet and would require tieback soil anchors or internal bracing for extra support of excavation heights greater than 16 feet.

Steel sheet piles could feasibly be installed to depths of 80 feet or greater at the Site provided the installation contractor was prepared to use a vibratory hammer and/or high pressure jetting at the toe of the piles to loosen the denser deep alluvium soils. Typically ‘Z-section’ steel sheet piles are used for deep excavations because of their high rigidity to weight ratio. Heavy duty sheet pile sections, such as an AZ50 section, may be required for the partial dewatering scenarios that intend on minimizing groundwater drawdown by maximizing the support elevations and load carrying capabilities of the sheet pile section.

Other alternatives for an impermeable shoring system include the use of a continuous or secant pile wall or a structural slurry wall system for support of the proposed excavations; however, these systems would require multiple installation components, would not be fully salvageable or re-usable, and appear to be less cost-effective for this application.

Tieback Anchors/Internal Bracing

Impermeable shoring systems for this application will be subject to both lateral earth pressures and unbalanced hydrostatic pressures and could require additional restraint through tieback anchors or internal braces to help support the excavations.

While a cantilever system may be adequate for the shallow excavation scenario, it may be more cost-effective to include at least one row of anchors or bracing. For the dry excavation scenarios extending to depths of 40 feet, at least three rows of tiebacks or internal bracing would be needed for additional support. Wet excavation scenarios extending to depths of 34 feet may only require two rows of tieback anchor supports depending on the amount of partial dewatering and location of the supports.

Tieback anchors are typically installed on 6- to 8-foot center-to-center spacing, cannot be re-used, but maintain an open excavation for easier access. For these preliminary studies, we recommend assuming the uppermost tieback anchor will be located at a minimum of 5 feet below the ground surface. Internal braces or struts can be installed on greater spacing (10- to 20-foot center-to-center), require a reaction source, can be re-used, but will span the interior of the excavation creating access issues. The reaction source for an internal brace system can be an adjacent shoring wall or deadmen at the base of the excavation.

Tieback anchors would be preferred if a single mass excavation is planned with shoring required around the perimeter only. Internal braces or struts would be more efficient for smaller, segmented excavations. An internal brace system can span an excavation width of up to 100 feet, but a raker system with struts directed to reaction deadmen anchors in the base of the excavation would be needed to span larger widths. A raker system could only be used in a "dry" excavation and given the construction interference it would cause, tiebacks again appear as the preferred alternative for preliminary analysis.

Shoring Analyses

Preliminary shoring analyses were completed with the aid of Shoring Suite v8.10g, a shoring analysis software program developed by CivilTech Software. Shoring Suite software can determine shoring size and embedment criteria as well as estimated moment, shear, and deflection of shoring systems. The Shoring Suite is based on methods and design data as presented by the U.S. Navy Design Manual DM-7, Steel Sheet Piling Design Manual (USS), and Federal Highway Design and Construction Summary (FHWA-RD-75). For the purposes of our analyses, the Shoring Suite software was used to develop preliminary and generalized shoring embedment and support criteria for the various scenarios.

Our analyses typically used a conservative representation of the Site subsurface conditions. As the remediation plan develops and specific areas are identified for excavation, we recommend more detailed and refined excavation and shoring analyses be completed for the individual areas.

Note, the basal stability of the proposed excavations will require a more thorough hydrogeologic analysis. Our preliminary study did not include detailed basal stability analyses and the embedment and support criteria provided below should be taken as minimums with the understanding that a detailed hydrogeologic analysis may result in deeper embedment criteria and groundwater cutoff requirements to prevent blowout at the base of the excavations.

Dry Excavation Scenarios

Two dry excavation scenarios, where dewatering below the base of the excavation for a dry work environment is assumed, were considered. In general deeper embedment of the shoring wall and/or more supports are required for the dry excavation scenarios to account for the unbalanced hydrostatic pressures created by the full dewatering of the excavation.

- **Shallow Excavation Scenario (20 Feet)**. An anchored sheet pile wall can be used with a minimum required embedment depth of 15 feet for a minimum total pile length of 35 feet and one row of tieback/strut support.
- **Deep Excavation Scenario (40 Feet).** An anchored sheet pile wall can be used with a minimum required embedment depth of 27 feet for a minimum total pile length of 67 feet with three rows of tiebacks/strut supports.

Wet Excavation Scenarios

Three wet excavation scenarios were considered. The goal of the wet excavation scenarios is to minimize the amount of dewatering associated with the shoring wall installation. The controlling feature to minimize dewatering is the location of the tieback anchor supports on the wall. Dewatering is assumed to be required to a minimum level of 3 feet below the lowest tieback anchor location to allow for a dry work environment during the installation of the anchors. Iterative analyses were performed to determine the required tieback anchor locations and associated amount of dewatering. The preliminary criteria for the three scenarios listed below include the assumption that a stiff sheet pile section, AZ50 or equivalent, will be used for the shoring wall. It is possible to locate the anchor supports higher on the shoring wall if a stiffer sheet pile section is used thereby reducing the required amount of dewatering.

- **Shallow Excavation Scenario (up to 16 Feet)**. A cantilever sheet pile wall can be used with a minimum embedment depth of approximately 35 feet for a minimum total pile length of 50 feet.
- **Moderately Deep Excavation Scenario (between 16 and 22 Feet).** An anchored sheet pile wall can be used with one row of tieback/strut supports and a minimum embedment depth ranging from 12 to 20 feet. The resulting total pile length will range from 27 to 42 feet, respectively.
- **Deep Excavation Scenario (between 22 and 34 Feet).** An anchored sheet pile wall can be used with two rows of tieback/strut supports and a minimum embedment depth ranging from 17 to 26 feet. The resulting total pile length will range from 39 to 60 feet, respectively.

Cost Estimate Considerations

The following cost information was derived from discussions with select local contractors and suppliers as well as the RSMeans Costworks database. These costs should be considered preliminary and we recommend adding a 25 to 30 percent contingency to these costs for estimating or budgeting purposes.

- **Shallow, Dry Excavation Scenario (20 Feet).** $61 per square foot of exposed wall at the end of excavation. Assumes an embedment depth of 15 feet for a total pile length of 35 feet and one row of tieback/strut support.

- **Deep, Dry Excavation Scenario (40 Feet).** $72 per square foot of exposed wall at the end of excavation. Assumes an embedment depth of 27 feet for total pile length of 67 feet with three rows of tiebacks/strut supports.
- **Wet Excavation Scenarios (10 to 34 Feet).** $92 per square foot of exposed wall at the end of excavation. Assumes embedment depths ranging from 12 to 35 feet for total pile lengths ranging from 27 to 60 feet with a maximum of two rows of tieback anchor supports. Assumes a stiff sheet pile section, AZ50 or equivalent, will be used.

The cost estimates above include the material costs, driving and removing/salvaging sheet piles, associated labor, and tieback/internal brace installation. Additional embedment of the sheet pile walls for purposes of cutting off groundwater or extending the excavation depths for the above scenarios would result in increased unit costs.

Limitations

Work for this project was performed and this memorandum prepared in accordance with generally accepted professional practices for the nature and conditions of work completed in the same or similar localities, at the time the work was performed. It is intended for the exclusive use of Aspect Consulting, LLC for specific application to the referenced project. This memorandum does not represent a legal opinion. No other warranty, expressed or implied, is made.

This memorandum is issued with the understanding that the information and considerations contained herein will be used as a basis for engineering design of the planned improvements.

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